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ANDERSON ENGINEERING INC SPRINGFIELD MO

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NATIONAL DAM SAFETY PROGRAM. DAM A-21 (MO 10144), MISSOURI - KA--ETC(U)

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## MISSOURI - KANSAS CITY BASIN

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DAM A-21

LAFAYETTE COUNTY, MISSOURI

MO 10144

# PHASE I INSPECTION REPORT NATIONAL DAM SAFETY PROGRAM

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PREPARED BY: U. S. ARMY ENGINEER DISTRICT, ST. LOUIS

FOR: STATE OF MISSOURI

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DECEMBER 1978

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REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER	2. GOVT ACCESSION NO. AD-A106	3. RECIPIENT'S CATALOG NUMBER 510
4. TITLE (and Subtitle) Phase I Dam Inspection Report National Dam Safety Program Dam A-21 (MO 10144) Lafayette County, Missouri		5. TYPE OF REPORT & PERIOD COVERED Final Report
7. AUTHOR(s) Anderson Engineering, Inc.		6. PERFORMING ORG. REPORT NUMBER
9. PERFORMING ORGANIZATION NAME AND ADDRESS U.S. Army Engineer District, St. Louis Dam Inventory and Inspection Section, LMSED-PD 210 Tucker Blvd., North, St. Louis, Mo. 63101		8. CONTRACT OR GRANT NUMBER(s) DACW43-78-C-0166
11. CONTROLLING OFFICE NAME AND ADDRESS U.S. Army Engineer District, St. Louis Dam Inventory and Inspection Section, LMSED-PD 210 Tucker Blvd., North, St. Louis, Mo. 63101		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS 1711
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office) National Dam Safety Program. Dam A-21 (MO 10144), Missouri - Kansas City Basin, Lafayette County, Missouri. Phase I Inspection Report.		12. REPORT DATE December 1978
16. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report) Approved for release; distribution unlimited.		13. NUMBER OF PAGES Approximately 55
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)		15. SECURITY CLASS. (of this report) UNCLASSIFIED
18. SUPPLEMENTARY NOTES		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE
19. KEY WORDS (Continue on reverse side if necessary and identify by block number) Dam Safety, Lake, Dam Inspection, Private Dams		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.		

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DEPARTMENT OF THE ARMY  
ST. LOUIS DISTRICT, CORPS OF ENGINEERS  
210 NORTH 12TH STREET  
ST. LOUIS, MISSOURI 63101

IN REPLY REFER TO

SUBJECT: Dam A-21 (Mo. 10144) Phase I Inspection Report

This report presents the results of field inspection and evaluation of Dam A-21 (Mo. 10144):

It was prepared under the National Program of Inspection of Non-Federal Dams.

This dam has been classified as unsafe, non-emergency by the St. Louis District as a result of the application of the following criteria:

- 1) Spillway will not pass 50 percent of the Probable Maximum Flood.
- 2) Overtopping could result in dam failure.
- 3) Dam failure significantly increases the hazard to loss of life downstream.

SIGNED

SUBMITTED BY:

Chief, Engineering Division

19 MAR 1979

Date

SIGNED

APPROVED BY:

Colonel, CE, District Engineer

20 MAR 1979

Date

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DAM A-21  
LAFAYETTE COUNTY, MISSOURI  
MISSOURI INVENTORY NO. 10144

PHASE I INSPECTION REPORT  
NATIONAL DAM SAFETY PROGRAM

Prepared By  
Anderson Engineering, Inc., Springfield, Missouri  
Hanson Engineers, Inc., Springfield, Illinois

For  
The Governor of Missouri

December, 1978

PHASE I REPORT  
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Dam A-21  
State Located: Missouri  
County Located: Lafayette County  
Stream: Unnamed Tributary to Big Sni-A-Bar Creek  
Date of Inspection: 3 August 1978

Dam A-21 was inspected by an interdisciplinary team of engineers from Anderson Engineering, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

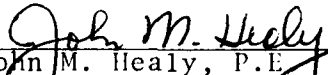
The guidelines used in the assessment were furnished by the Department of the Army, Office of the Chief of Engineers and they have been developed with the help of several Federal and State agencies, professional engineering organizations, and private engineers. Based on these guidelines, this dam has been classified by the St. Louis District Corps of Engineers as an intermediate size dam with a high downstream hazard potential. Their estimate of the damage zone extends 4 miles downstream of the dam. Within the damage zone are seven houses, two farm complexes, one business, one unimproved road crossing, one improved road bridge, one U.S. highway bridge and one railroad bridge.

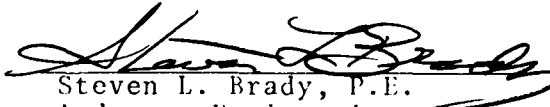
Our inspection and evaluation indicates that the combined spillways do not meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. The combined spillways will pass 33 percent of the Probable Maximum Flood without overtopping. The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The guidelines require

that a dam of intermediate size with a high downstream hazard potential pass 100 percent of the PMF without overtopping. Fifty percent of the PMF was also routed through the spillways resulting in a dam overtopping of 0.83 ft. The combined spillways will pass a 100 year flood event, without overtopping. A 100 year flood is one that has a 1 percent chance of being exceeded in any given year.

The embankment and appurtenances are generally in good condition. Deficiencies, including erosion and tree growth were noted and should be corrected by the owner. Wet areas were noted on the lower berm on the downstream face. It could not be determined whether these areas represented seepage or just poor drainage. Further evaluation of this condition was recommended after the downstream face is cleared of overgrowth. Another deficiency was the lack of seepage analysis records.

A detailed report is attached to be submitted to the owners and to the Governor of Missouri.

  
John M. Healy, P.E.  
Hanson Engineers, Inc.

  
Steven L. Brady, P.E.  
Anderson Engineering, Inc.

PHASE 1 INSPECTION REPORT  
NATIONAL DAM SAFETY PROGRAM  
DAM A-21 - ID NO. 10144

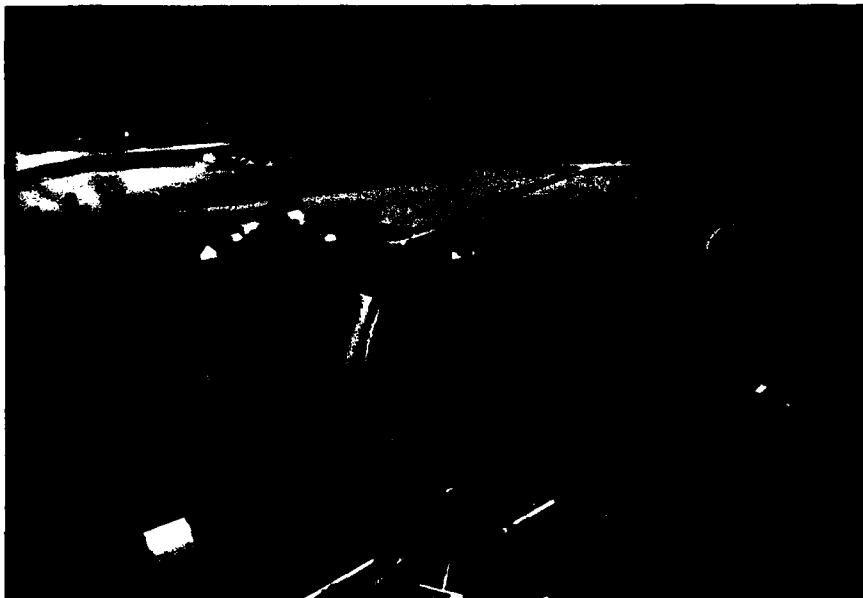
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Aerial View of Dam and Lake

## SECTION 1 - PROJECT INFORMATION

### 1.1 GENERAL:

#### A. Authority:

The National Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the St. Louis District, Corps of Engineers, District Engineer directed that a safety inspection of Dam A-21 in Lafayette County, Missouri be made.

#### B. Purpose of Inspection:

The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and a visual inspection in order to determine if the dam poses hazards to human life or property.

#### C. Evaluation Criteria:

Criteria used to evaluate the dam were furnished by the Department of the Army, Office of the Chief of Engineers, "Recommended Guidelines For Safety Inspection of Dams." These guidelines were developed with the help of several federal agencies and many state agencies, professional engineering organizations, and private engineers.

### 1.2 DESCRIPTION OF PROJECT:

#### A. Description of Dam and Appurtenances:

Dam A-21 is an earth fill structure approximately 52 ft high and 520 ft long at the crest. The appurtenant works consist of a concrete drop inlet and reinforced concrete pipe primary spillway, which is located near the center of the dam, and a grass covered emergency spillway, which is located at the north abutment. Sheet 4 of Appendix A shows a plan of the embankment and spillways and a typical section of the embankment.

#### B. Location:

The dam is located in the northwest part of Lafayette County, Missouri on a small tributary of Big Sni-A-Bar Creek. The lake formed by the dam is shown on the Lexington West, Camden, Bates City and Odessa North, Missouri quadrangle

maps in the SW 1/4 of the NW 1/4 of Section 21, T50N, R28W. Sheet 1 of Appendix A shows the general vicinity and Sheets 2 and 3 of Appendix A shows a plan of the immediate area of the dam and lake.

C. Size Classification:

With an embankment height of 52 ft and a maximum storage capacity of approximately 485 acre-ft, the dam is in the intermediate size category.

D. Hazard Classification:

The St. Louis District, Corps of Engineers has classified this dam as a high hazard dam. Their estimate of the damage zone extends 4 miles downstream of the dam. Within the damage zone are seven houses, two farm complexes, one business, one unimproved road crossing, one improved road bridge, one U.S. highway bridge and one railroad bridge.

E. Ownership:

The dam was designed by the Soil Conservation Service (SCS) but the property upon which the dam and lake are located is retained by the property owner or owners. The As-Built plans indicate the primary owners to be Robert Riesmeyer, Raymond Wieligman and Mamie Breuer. These owners granted an easement to the Wellington-Napolean Watershed Subdistrict to construct, operate, and maintain this structure. The subdistrict is the owner and is responsible for the structure. The address of the Subdistrict is 120 W 19th Street, Higginsville, Missouri 64037.

F. Purpose of Dam:

The purpose of this dam according to the PL-566 watershed program is to provide watershed protection and flood prevention. The purpose of these structures is for grade stabilization with flood water retarding features. These lakes may be stocked with fish but not by the Soil Conservation Service. They may be stocked with fish from the Federal and State Fisheries in cooperation with individual landowners.

G. Design and Construction History:

The dam was designed by the Soil Conservation Service and constructed under their inspection supervision (inspection handled by the Higginsville District Office). The

dam was completed in 1966. As-Built plans are available and have been used to prepare this report. No significant problems in regards to seepage through or stability of the embankment are reported to have occurred since the dam was built. According to SCS district personnel, no modifications have been made to the dam.

#### H. Normal Operating Procedure:

Normal flows will be passed by an uncontrolled drop inlet spillway, whereas a grassed emergency spillway would come into operation for major floods. A local resident indicated that the maximum water depth ever experienced was 1 ft to 2 ft above the primary spillway crest in 1973. The emergency spillway has apparently never come into service.

#### 1.3 PERTINENT DATA:

Pertinent data about the dam, appurtenant works, and reservoir are presented in the following paragraphs. Sheet 4 of Appendix A is a plan of the embankment and spillways with a typical cross section of the dam. Sheet 5 presents a plan and profile of the primary spillway. Sheet 6 presents a profile and cross section of the foundation drainage system.

#### A. Drainage Area:

The drainage area for this dam, as obtained from the As-Built plans, is equal to approximately 426 acres.

#### B. Elevations (Feet Above M.S.L.):

- (1) Top of dam (measured): north end 768.0; center 768.5;  
south end 768.0.  
Top of Dam (As-Built Plans): north end 767.7;  
center 769.0; south end 767.7.
- (2) Principal Spillway Crest: As-Built Plans 762.0;  
measured 761.5.
- (3) Emergency Spillway Crest: As-Built Plans 764.5;  
measured 764.2.

- (4) Primary Spillway Outlet Pipe Invert: As-Built Plans 719.0; measured 719.0.
- (5) Maximum Design Pool (As Built Plans): 767.2.
- (6) Pool on Date of Inspection: measured 761.7.
- (7) Streambed at Primary Spillway Outlet: As-Built Plans 715.5; measured 714.1.
- (8) Maximum Tailwater: Unknown.

C. Discharge at Dam Site:

- (1) All discharge at the dam site is through uncontrolled spillways.
- (2) Estimated Discharge Capacity at Top of Dam (El. 768.0): 3246 cfs.

D. Reservoir Surface Areas:

- (1) At Principal Spillway Crest: As Built Plans 20.8 acres.
- (2) At Top of Dam: As Built Plans 28.8 acres.

E. Storage Capacities:

- (1) At Principal Spillway Crest: 543 acre-ft.
- (2) At Top of Dam (El. 768.0): 485 acre-ft.

F. Reservoir Lengths:

- (1) At Principal Spillway Crest (Estimated from As-Built Plans): 5300 ft.
- (2) At Top of Dam (Estimated from As-Built Plans): 3500 ft.

G. Dam:

- (1) Type: rolled earth.
- (2) Length at Crest: 520 ft.
- (3) Height: 52 ft.

- (4) Top Width: 14 ft.
- (5) Side Slopes: 2.5 H: 1 V.
- (6) Zoning: homogeneous silts and clays.
- (7) Cutoff: shallow core trench.

#### H. Principal Spillway:

- (1) Location: center of dam--Station 4+50
- (2) Type: 2 ft by 6 ft drop inlet concrete structure (crest elevation 761.5, 12 ft in length) with a 24 in. diameter reinforced concrete outlet pipe through the dam. The outlet pipe is 204 ft long, supported on a type A3 cradle with 5 concrete antiseep collars. The pipe inlet invert is at El. 750.00 and the outlet invert is at El. 719.00 (see Sheet 5 Appendix A). A plunge pool has been created at the end of the outlet pipe and dissipates the energy of the flow.

#### I. Emergency Spillway:

- (1) Location: south abutment.
- (2) Type: grass covered earth with 50 ft crest length and 3 H: 1 V side slopes.

## SECTION 2 - ENGINEERING DATA

### 2.1 GENERAL:

Available design computations and reports for Dam A-21 include a geology and soils report which contains soils testing information for the foundation and borrow materials (includes soil classifications, grain size analyses, shear strength tests, consolidation tests and permeability tests). Based on this information, design recommendations were made regarding site preparation, foundation drainage and embankment configurations. The As-Built plans contain a summary of the hydrologic and hydraulic design data used for the primary and emergency spillways. No documentation of construction inspection records have been obtained. There are no documented maintenance and operation data to our knowledge.

### 2.2 DESIGN:

#### A. Surveys:

The As-Built drawings show the topography of the immediate dam site area (Sheet 4 of Appendix A). A bench mark in the form of a bolt in the center of a bronze plate is located on the outlet end of spillway (BM No. 2 - Elev. = 760.05).

#### B. Geology and Subsurface Materials:

Physiographically, the site is located in the Missouri River loess hills area, which is characterized by gently rolling topography. The subsurface materials in upland areas generally consist of in excess of 20 ft of loess underlain by a Kansan Age glacial till material. Geological maps of the area indicate that the bedrock is the Marmaton group of the upper Des Moinesian series of the Pennsylvanian system. The Marmaton group consists of a succession of shale, limestone, clay and coal beds.

A publication entitled "Evaluation of Missouri's Coal Resources" by the Missouri Geological Survey indicates that the "Lexington Coal Bed" was mined extensively in this area. The maps associated with this publication indicate that the dam site lies near the southern boundary of the undermining activity and that the coal seam mined was approximately 20 in. thick in the area. The U.S.G.S. quad sheet for the area



(Camden, Missouri, 1950) indicates an inactive mine shaft approximately one mile northwest of the dam (see Sheet 1 of Appendix C). The Coal Resources publication previously mentioned indicates that the depth to the coal seam at that location is approximately 32 ft and that the thickness of the seam is 18 in. If the coal seam is horizontal, then it would be at a depth of approximately 50 ft below the stream bed at the center of the dam (coal seam at elevation 660 to 665).

A boring plan and description of the soils encountered in the borings (Sheets 14 and 15 of the As-Built plans) are presented as Sheets 1 and 2 of Appendix B. Sheets 3, 4 and 5 of Appendix B present a description of the surface geology and physiography, and interpretations and conclusions regarding the soils encountered in the boring program (from geology and soils report by SCS). The soils encountered in the borings are generally low plasticity clays and silts. Dry density determinations on "core" samples were between 1.04 g/cc (64.8 pcf) and 1.39 g/cc (86.7 pcf), and estimated "blow counts" were between 5 and 10. Sand layers (1 ft to 3 ft thick) were encountered in several of the borings between elevations 700 and 710. One deeper sand layer (7 ft thick) was encountered in boring 303 at about elevation 690. "Refusal" was encountered in borings 6 and 301 at elevation 675 to 680 (probable elevation of bedrock).

#### C. Foundation and Embankment Design:

Reference should be made to Sheets 6 through 9 of Appendix B which contain a summary of the soil test data and recommendations for the foundation and embankment design (from geology and soils report by SCS). Because of the existence of sand layers a foundation drainage system was developed (includes a drainage trench and vertical drains penetrating to elevation 700). The foundation drainage system is shown on Sheets 5 and 6 of Appendix A (from As-Built Plans). A shallow core trench apparently was constructed at the base of the dam along its entire length.

An abutment drain was constructed in the south abutment as shown on Sheets 5 and 6 of Appendix A. This was apparently not part of the original design but may have been required to control ground water or "springs" during construction. The landowner mentioned that several "springs" existed in the stream valley before the dam was built.

Borrow material for the dam was obtained from the reservoir area upstream of the embankment. Stability analyses based on the use of this material were performed by SCS. It was recommended that the embankment materials be compacted to 95 percent of the maximum dry density as obtained by the Standard Proctor Compaction Test and at a moisture content wet of optimum. There is apparently no particular zoning of the embankment, and no internal drainage features (except for the previously described foundation drainage system) are known to exist. No construction inspection test results have been obtained.

#### D. Hydrology and Hydraulics:

Design data, storage curves and routing curves for the "emergency spillway" and "freeboard" hydrographs are presented on Sheets 10 through 12 of Appendix B (from As-Built plans by SCS). Based on this data, a field check of spillway dimensions and embankment elevations, and a check of the drainage area on U.S.G.S. quad sheets, a hydrologic analysis using U.S. Army Corps of Engineers guidelines was performed and appears in Appendix C, Sheets 1 to 8. It was concluded that the primary and emergency spillways combined will pass 33 percent of the Probable Maximum Flood.

#### E. Structure:

Structural design computations for appurtenant structures were not obtained. Details of all concrete structural elements (riser structure, etc.) are shown on the As-Built plans.

#### 2.3 CONSTRUCTION:

No construction inspection data has been obtained. Construction supervision was accomplished by the Soil Conservation Service district office in Higginsville, Missouri.

#### 2.4 OPERATION AND MAINTENANCE:

On this structure, there is an operation and maintenance agreement between the Soil Conservation Service and the Wellington-Napolean Watershed Subdistrict. The operation and maintenance agreement spells out the operation and maintenance requirements and the inspection procedures. District SCS office personnel indicated that a yearly questionnaire is sent to land owners inquiring as to maintenance problems. It was reported that inspection stops are made on an irregular basis by SCS district personnel (Higginsville office).

#### 2.5 EVALUATION:

Slope stability analyses were performed, but seepage analyses comparable to the Recommended Guidelines were not available. The foundation drainage system shown on Sheet 6 of Appendix A would indicate that a seepage analysis has been performed. The owner should either locate these analyses or have them performed by an engineer experienced in the design of dams.

The engineering data available were inadequate to make a detailed assessment of the design and particularly the construction of the dam. No valid engineering data on the construction of the embankment were found.

## SECTION 3 - VISUAL INSPECTION

### 3.1 GENERAL:

The field inspection was made on 3 August 1978. The inspection team consisted of personnel from Anderson Engineers, Inc. of Springfield, Missouri and Hanson Engineers, Inc. of Springfield, Illinois. The team members were:

Mike Gray - Anderson Engineers  
(Instrument Man)

Steve Brady - Anderson Engineers  
(Civil Engineer)

Jack Healy - Hanson Engineers  
(Geotechnical & Structural Engineer)

Gene Wertepny - Hanson Engineers  
(Hydraulics Engineer)

Dave Daniels - Hanson Engineers  
(Geotechnical & Hydraulics Engineer)

### 3.2 DAM:

The dam is an earth fill embankment constructed from borrow obtained from the emergency spillway area and the reservoir area below normal pool. Based on the soil borings, the fill material would be expected to consist of low plasticity clays and silts.

The embankment appeared to be in generally good condition except for the following deficiencies: (1) Slight erosion on upstream face 50 ft north of primary spillway; (2) Erosion channels on downstream abutment-dam contacts (both abutments) primarily from upper berm downward; (3) Scattered growth of locust trees up to 5 in. in diameter on the downstream face.

No sloughing of the embankment or seepage through or under the embankment was evident. The foundation drain outlet was flowing (estimated flow less than 0.5 gpm). The south abutment drain was also flowing (estimated flow less than 1 gpm).

Some water was standing in the middle of the lower berm on the downstream side of the dam. Because of high grass and brush, it could not be determined whether this was seepage water or merely due to poor drainage of the inward sloping berm.

The horizontal alignment appeared as constructed. No surface cracking or unusual movement was obvious. It should be noted, however, that elevations of the primary spillway crest and the center of the dam which were obtained in the field were approximately 0.5 ft lower than as indicated on the As-Built Plans (see Section 1.3.B of this report). All other elevations obtained in the field agreed fairly well with those indicated on the As-Built Plans. The discrepancy at the center of the dam might be explained by the possibility of some post construction settlement of the center portion of the dam.

No instrumentation (monuments, piezometers, etc.) were observed.

#### A. Primary Spillway and Outlet:

The riser structure was in good condition--no cracking or spalling of concrete was noted. The intake structure was surrounded by heavy grass and some wood debris. Water was flowing into the riser structure on the day of inspection.

The outlet pipe was also in good condition. The flow from the outlet pipe was estimated to be less than 1 cfs. A plunge pool has been created at the end of the outlet pipe and acts as an energy dissipator. A 4 ft deep erosion channel was noted along the south side of the primary spillway outlet pipe.

The outlet channel is overgrown with small trees and brush near the primary spillway outlet pipe. The outlet channel side slopes are in good condition except for one small slough 20 to 30 ft downstream on the north bank.

Along the last portion of the primary spillway outlet pipe, there is a 6 in. diameter corrugated iron pipe (Class II, Type A), which is the outlet of the foundation drainage system for the embankment. The pipe has a length of 80 ft and a slope of 0.010. It is shown on Sheets 5 and 6 of Appendix A.

B. Emergency Spillway:

The emergency spillway is in good condition. It measures 50 ft in width with 3 H: 1 V side slopes. The base and side slopes of the emergency spillway are grass covered. No erosion was noted and it appears that the emergency spillway has never been used.

3.3 RESERVOIR AND WATERSHED:

The immediate periphery of the lake was grass covered with moderate slopes. No sloughing of the reservoir banks were noted. A few areas of minor erosion were noted.

3.4 EVALUATION:

Tree growths noted on the dam should be removed and all future growth should be removed on a yearly basis. Grass should be cut and debris should be removed from around the primary spillway crest. Excessive growths and debris in this area could cause entrance restrictions. Visually observed erosional areas are deficiencies which, if left uncontrolled or uncorrected, could lead to serious problems in the future. These deficiencies should be able to be corrected by normally scheduled routine maintenance. The wet areas noted on the lower berm should be investigated more thoroughly after excess brush and overgrowth is removed to be sure that these areas are not associated with seepage through the dam.

Photographs of the dam, appurtenant structures, and the reservoir and watershed are presented in Appendix D.

## SECTION 4 - OPERATIONAL PROCEDURES

### 4.1 PROCEDURES:

There are no controlled outlet works for this dam; therefore, no regulating procedures exist. The pool is controlled by rainfall, runoff, evaporation and the capacities of the uncontrolled spillways.

### 4.2 MAINTENANCE OF DAM:

Based on the amount of brush and the size of trees on the downstream slope it has been several years since the vegetation on the dam has been cut. Maintenance in terms of tree and brush removal and mowing of the grass is apparently the responsibility of the land owner. A yearly questionnaire is sent to land owners inquiring as to maintenance problems. Inspection stops are reported to be made on an irregular basis by SCS district personnel.

### 4.3 MAINTENANCE OF OPERATING FACILITIES:

No operating facilities exist at this dam.

### 4.4 DESCRIPTION OF ANY WARNING SYSTEM AND AFFECT:

The inspection team is unaware of any existing warning system for this dam.

### 4.5 EVALUATION:

Tree and brush growth should be removed from the dam on a yearly basis. Erosional areas at abutment-dam contacts and other areas should be repaired. The use of riprap to prevent future erosion in these areas is a possibility.

## SECTION 5 - HYDRAULIC/HYDROLOGIC

### 5.1 EVALUATION OF FEATURES:

#### A. Design and Experience Data:

Design data used by the Soil Conservation Service to design this dam are shown on the As-Built plans and presented as Sheets 10 through 12 of Appendix B of this report. Based on this information, a field check of spillway dimensions and embankment elevations, and a check of the pool and drainage areas from U.S.G.S. quad sheets, a hydrologic analysis using U.S. Army Corps of Engineers guidelines was performed and appears in Appendix C, Sheets 1 to 8.

#### B. Visual Observations:

The riser structure and outlet pipe for the primary spillway appear in good condition. The earth and grass covered emergency spillway is in good condition. The primary spillway was flowing on the day of inspection (estimated flow less than 1 cfs). The emergency spillway has apparently never been used. A 4 ft deep erosion channel was noted along the south side of the primary spillway outlet pipe. A plunge pool has been created at the end of the primary spillway outlet pipe to dissipate the energy. The outlet channel is overgrown with small trees and brush.

No facilities are available to draw down the pool. The primary spillway is located near the center of the dam and the emergency spillway is located on the south abutment. Spillway releases would not be expected to endanger the integrity of the dam.

#### C. Overtopping Potential:

Based on the hydrologic and hydraulic analysis as presented in Appendix C, the combined primary and emergency spillways will not pass the Probable Maximum Flood without overtopping. The Probable Maximum Flood (PMF) is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The recommended guidelines from the Department of the Army, Office of the Chief of Engineers, require that this structure (intermediate size with high downstream hazard potential) pass 100 percent of the PMF, without overtopping.



The routing of the PMF through the spillways and Dam, indicated that the Dam will be overtopped by 1.99 ft at reservoir elevation 769.99. The duration of the overtopping will be 4.08 hrs. and the maximum outflow 6744 cfs. Fifty percent of the PMF was also routed through the spillways, resulting in a maximum reservoir elevation of 768.83, 0.83 ft above the top of the dam (768.00). The peak outflow was 2879 cfs. The portion of the PMF that will just reach the top of the dam is about 53 percent. The spillway's system will be able to pass the 100 year frequency flood without overtopping.

## SECTION 6 - STRUCTURAL STABILITY

### 6.1 EVALUATION OF STRUCTURAL STABILITY:

#### A. Visual Observations:

No serious deficiencies which would affect the structural stability of this dam were noted during the field inspection. However, if left unchecked, tree growth and the erosion at abutment-dam contact areas could cause stability problems in the future. The wet areas noted on the lower berm of the downstream face (Section 3.2) should be checked after the overgrowth is removed to be sure that this is not associated with a seepage condition through the dam.

#### B. Design and Construction Data:

Stability analyses were performed by the Soil Conservation Service and recommendations were made regarding side slopes, berm widths and compaction densities (see Sheets 6 through 9 of Appendix B). Our site inspection indicated that the side slopes and berm widths were as recommended. If the embankment was placed in relatively thin lifts at the recommended density of 95 percent of the Standard Proctor maximum dry density (no laboratory testing records available to verify this), then the embankment should remain stable.

A seepage analysis comparable to the requirements of the guidelines was not available, which is considered a deficiency and should be corrected.

#### C. Operating Records:

No appurtenant structures requiring operation exist at this dam.

#### D. Post-Construction Changes:

To our knowledge, no post-construction changes have been made.

#### E. Seismic Stability:

Considering the seismic zone (II) in which this dam is located, an earthquake of this magnitude is not expected to cause a structural failure to this dam.

## SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

### 7.1 DAM ASSESSMENT:

#### A. General:

This Phase I inspection and evaluation should not be considered as being comprehensive since the scope of work contracted for is far less detailed than would be required for an in-depth evaluation of dams. Latent deficiencies, which might be detected by a totally comprehensive investigation, could exist.

#### B. Safety:

The embankment appeared to be in generally good condition except for the following deficiencies: (1) Slight erosion on upstream face 50 ft north of primary spillway; (2) Erosion channels on downstream abutment-dam contacts (both abutments) primarily from upper berm downward; (3) A 4 ft deep erosion channel along the south side of the primary spillway outlet pipe; (4) Scattered growth of locust trees up to 5 in. in diameter on the downstream face; (5) overgrowth of trees and brush near the primary spillway outlet pipe; and (6) Lack of available seepage analyses.

Wet areas were noted in the middle of the lower berm on the downstream side of the dam. Because of the high grass and overgrowth, it could not be determined for certain whether these areas were associated with seepage through the dam or were merely the result of poor drainage (inward sloping berm, high grass, etc). This condition should be evaluated again after the trees and overgrowth have been removed.

The existing spillway system is inadequate to pass the PMF without overtopping the embankment. The dam will be overtopped by flows in excess of 33 percent of the Probable Maximum Flood. The Probable Maximum Flood (PMF) is defined as the flood discharge that may be expected from the most severe combination of meteorologic and hydrologic conditions that are reasonably possible in the region. Overtopping of an earthen embankment could cause serious erosion and could possibly lead to failure of the structure. The existing spillway system will be able to pass the 100 year frequency flood without overtopping.

C. Adequacy of Information:

The conclusions in this report were based on review of the As-Built plans, the geologic and soil mechanics report prepared by the Soil Conservation Service, the performance history as related by others, and visual observation of external conditions. The inspection team considers that these data are sufficient to support the conclusions herein.

D. Urgency:

The remedial measures recommended in paragraph 7.2 should be accomplished in the near future. If the deficiencies listed in paragraph B are not corrected and if good maintenance is not provided, the embankment condition will continue to deteriorate and it could become serious in the future. Top priority should be given to correcting inadequate spillways.

E. Necessity for Phase II:

Based on the result of the Phase I inspection, no Phase II inspection is recommended.

F. Seismic Stability:

This dam is located in Seismic Zone 1. An earthquake of this magnitude is not expected to be hazardous to this dam.

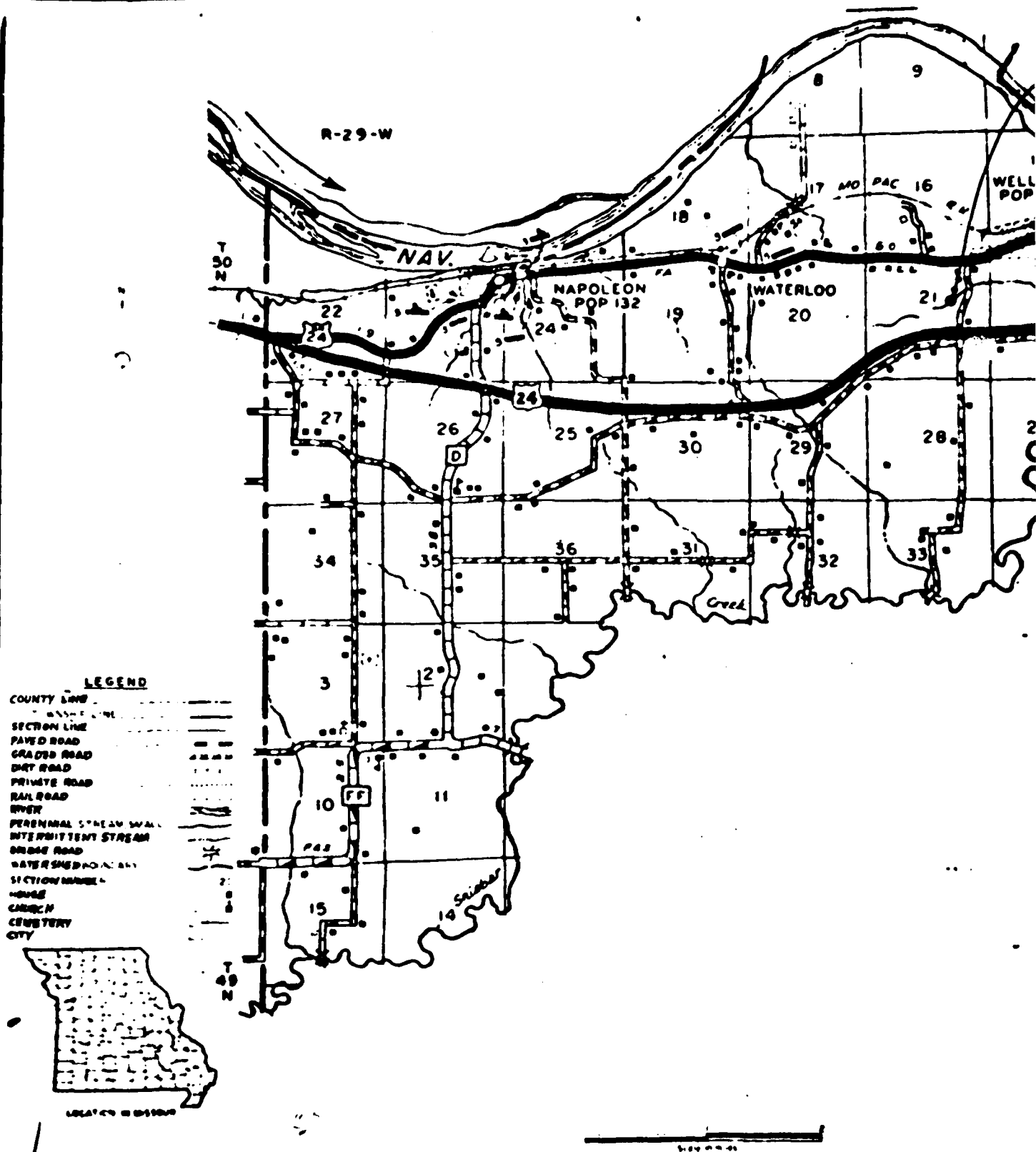
7.2 REMEDIAL MEASURES:

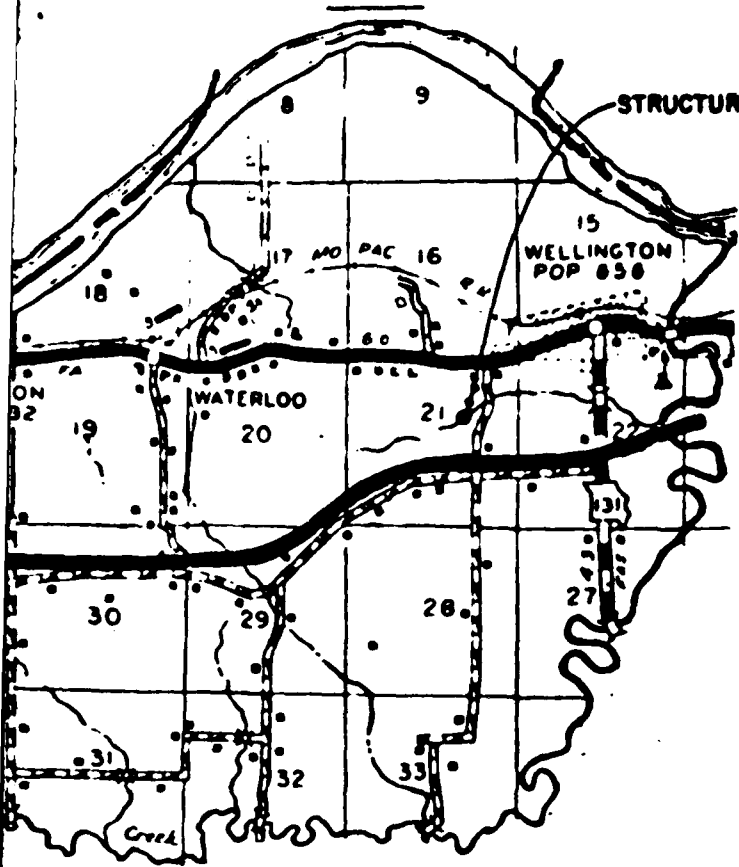
The following remedial measures and maintenance procedures are recommended and should be performed under the guidance of a professional engineer experienced in the design and construction of dams.

- (1) Spillway size and/or height of dam should be increased to pass the PMF. In either case, the spillway should be protected to prevent erosion.
- (2) Seepage analyses comparable to the requirements of the guidelines were not available, which is considered a deficiency and should be corrected.

- (3) Remove existing tree growth on the downstream face of the dam and remove all future tree and brush growth on a yearly basis. Cut the high grass and remove the debris around the primary spillway to prevent restrictions.
- (4) Correct the erosion activity at the embankment-abutment contacts on the downstream side of the dam and along the south side of the primary spillway outlet and place riprap in these areas to minimize erosion in the future.
- (5) Clear the outlet channel of the brush and tree growth for a distance of at least 50 ft beyond the end of the outlet pipe.
- (6) Check the downstream slope periodically for seepage and stability problems. If wet areas or seepage flows are observed, or if sloughing is noted, then the dam should be inspected and the situation evaluated by an engineer experienced in design and construction of dams.
- (7) A detailed inspection of the dam should be made at least every 5 years by an engineer experienced in the design and construction of dams. More frequent inspections may be required if slides, seeps, or other items of distress are observed.

APPENDIX A





INDEX OF DRAWINGS	
TYPE	SHEET NO.
Index Sheet	1
General Map of Reservoir	2
General Map of Reservoir	3
Plan of Embankment and Spillways	4
General Layout	5
Foundation Drainage System	6
Pipe Requirements	7
Structural Details	8
Architectural Details	9
Light Details	10
Truck Rack Details	11
Fence Details	12
Structural Details	13
Plan of Geologic Investigation	14
1000 Sections (Geologic Investigation)	15

U S DEPARTMENT OF AGRICULTURE  
SOIL CONSERVATION SERVICE

# DETAIL PLANS FOR WELLINGTON-NAPOLEON WATERSHED PROTECTION AND FLOOD PREVENTION PROJECT

LAFAYETTE COUNTY, MISSOURI  
IN COOPERATION WITH  
SOIL AND WATER CONSERVATION DISTRICT  
OF LAFAYETTE COUNTY  
LAFAYETTE COUNTY COURT

## STRUCTURE A-21

AS BUILT

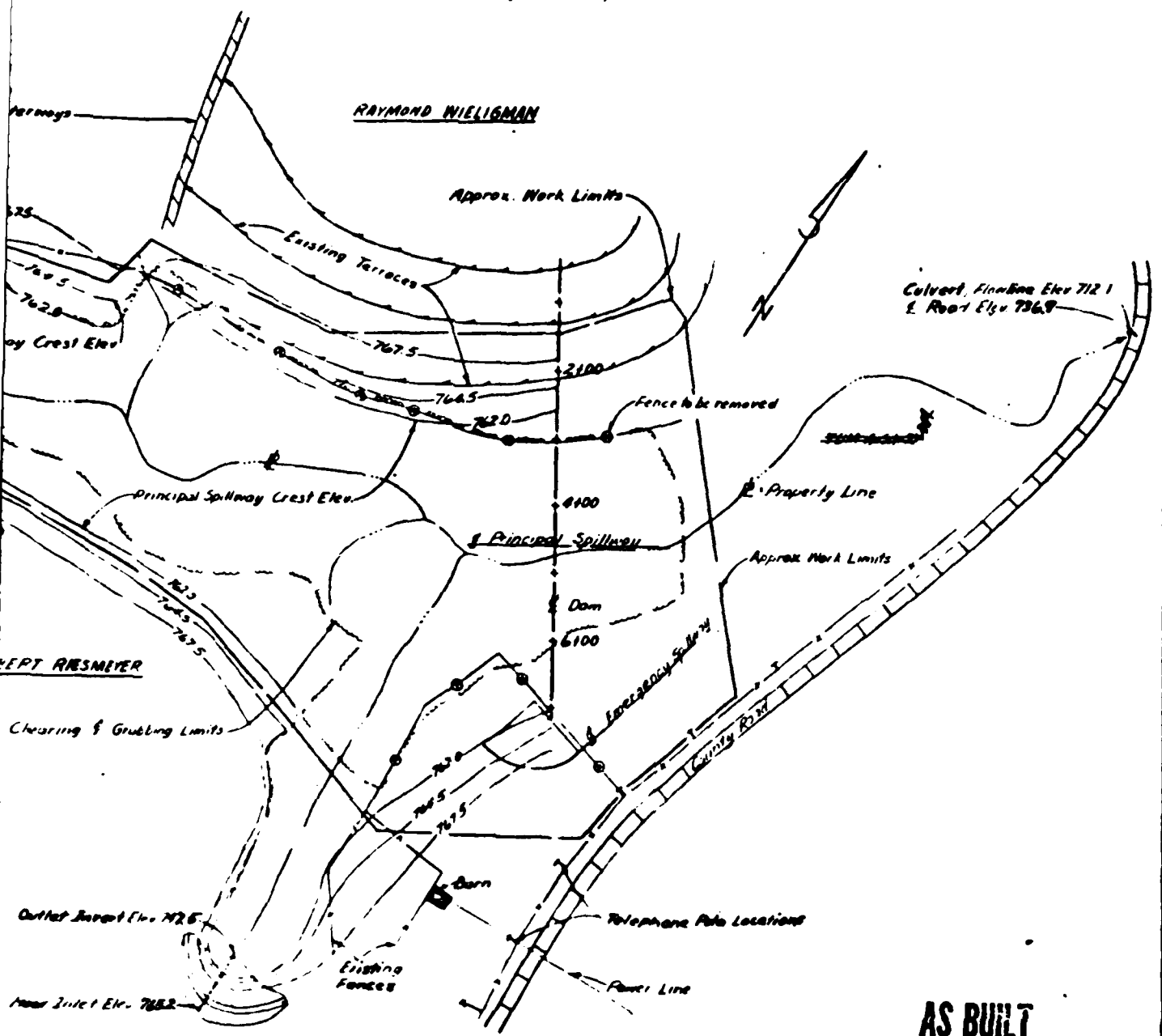
<i>James H. Smith</i>	5-6-66
<i>Ray L. Miller</i>	5-10-66

SHEET 1 APPENDIX A





~~FROM A-21-2 Elev 737.65, Gravel pits in south end of all pits and trees approx 236 upstream from actual right bank of stream~~



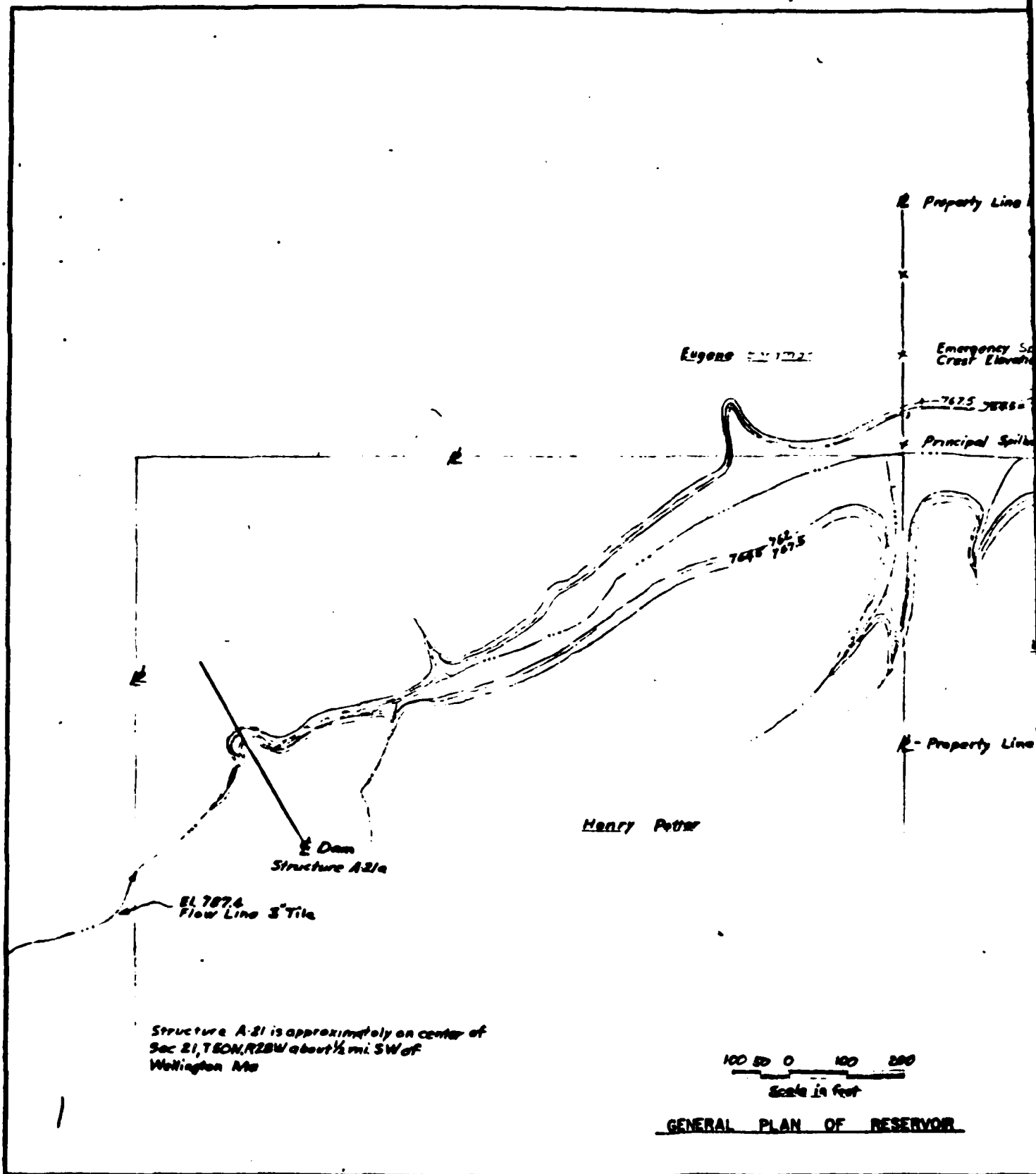
GENERAL PLAN OF RESERVOIR



2

**AS BUILT**

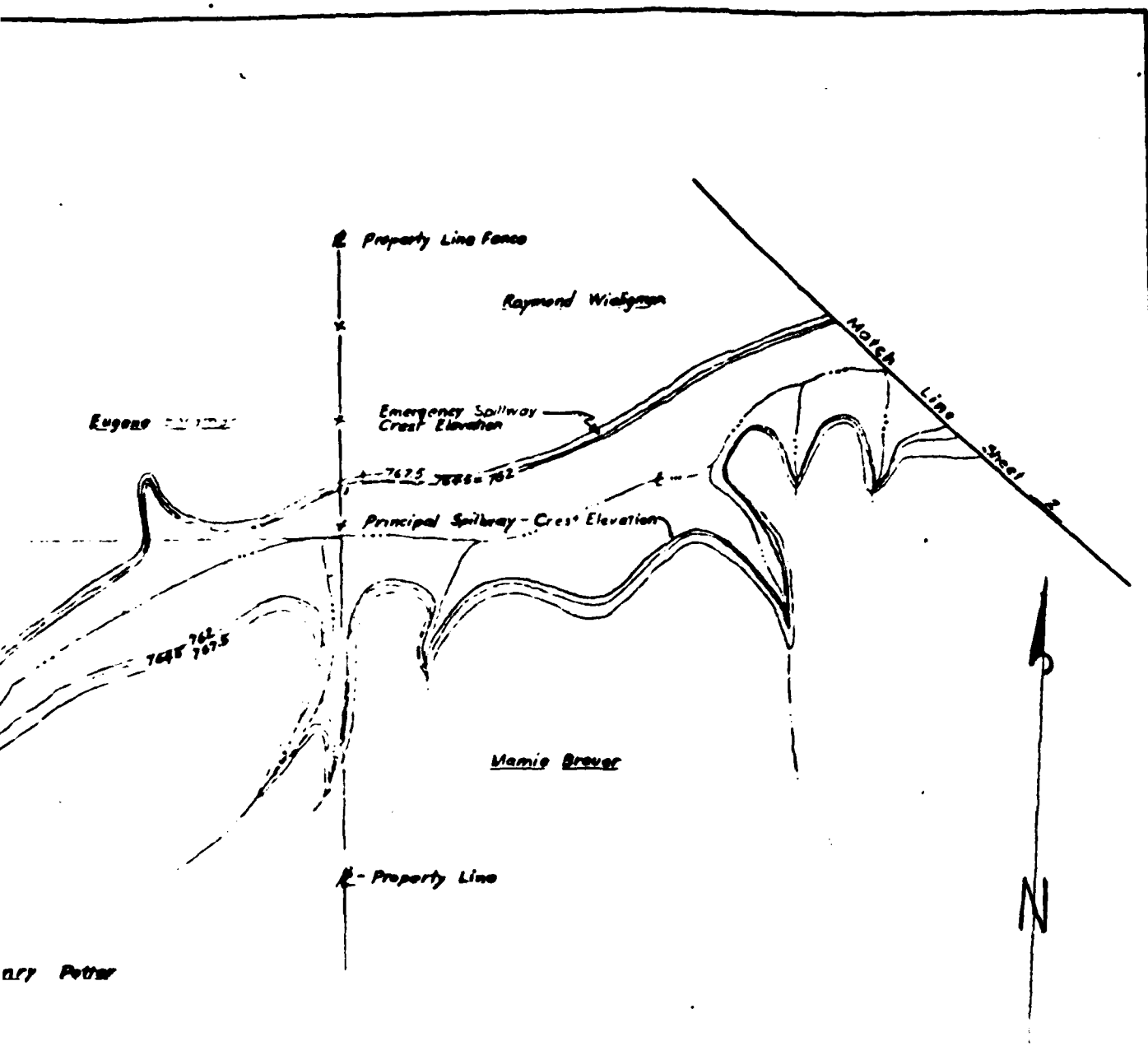
STRUCTURE A-21			
WELLINGTON-LAFAYETTE WATERSHED ELESS			
LAFAYETTE COUNTY, MISSOURI			
U. S. DEPARTMENT OF AGRICULTURE			
SOIL CONSERVATION SERVICE			
Project A E 10-11	5.26	1/2	1/2
Project A E 10-11	5.40	1/2	1/2
Project B E 5-11	1/84	1/2	1/2
Project C E 10-11	1/84	1/2	1/2
			5,6-23,179



Structure A-21 is approximately on center of  
Sec 21, T80N, R28W about 1/2 mi SW of  
Wellington Mo

100 50 0 100 200  
Scale in feet

GENERAL PLAN OF RESERVOIR



100 200 0 100 200  
Scale in feet  
GENERAL PLAN OF RESERVOIR

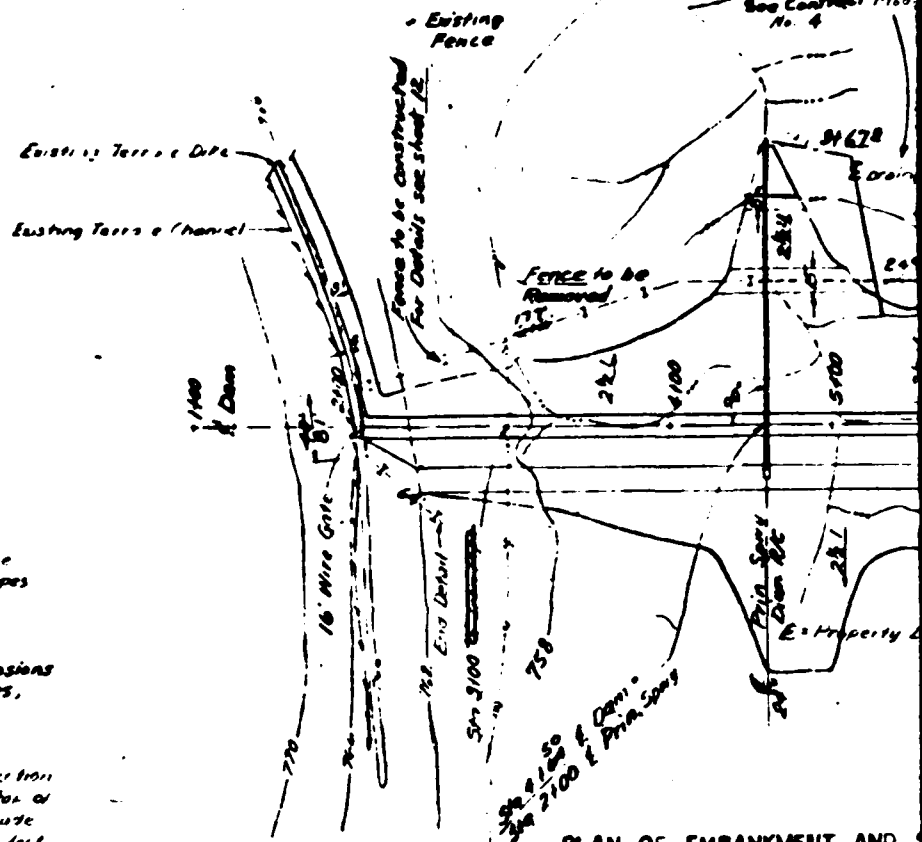
**AS BUILT**

PL 566 STRUCTURE A-21	
WELLINGTON-NAPOLEON WATERSHED	
LAFAYETTE COUNTY, MISSOURI	
U.S. DEPARTMENT OF AGRICULTURE	
SOIL CONSERVATION SERVICE	
A. E. Larkin	5/85
J. A. Mummel	7/85
M. H. Randall	2/00
56-25,179	

KAYMOVL, W. E. L. I. N. A. V.

ADAM W. WEIGMAN

ROBERT R COMEY



NOTES: Thompson?

A minimum of 6 inches topsoil to be placed on all compacted earth fill slopes

Accessions

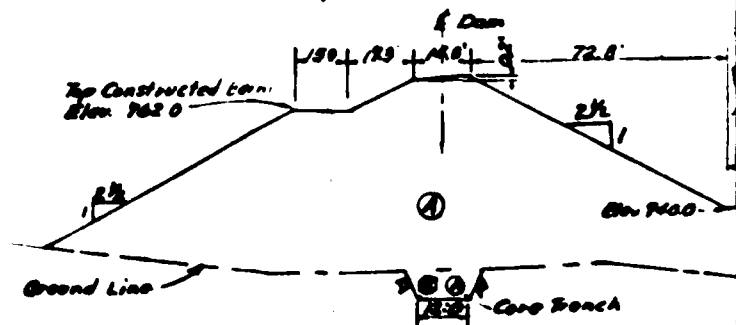
Emergency Spillway Diversion Dimensions  
18' effective height, 3:1 side slopes,  
minimum base width 12'

Existing Terrace Pile

Construct a concrete dike to cross section shown. At the dam the elevation of top of dike shall be 767.7. The dike will grade out at elevation 767.5, approx 165 feet downstream from E dam. Thus it is to be subsidiary to "Earth Fill, Class A".

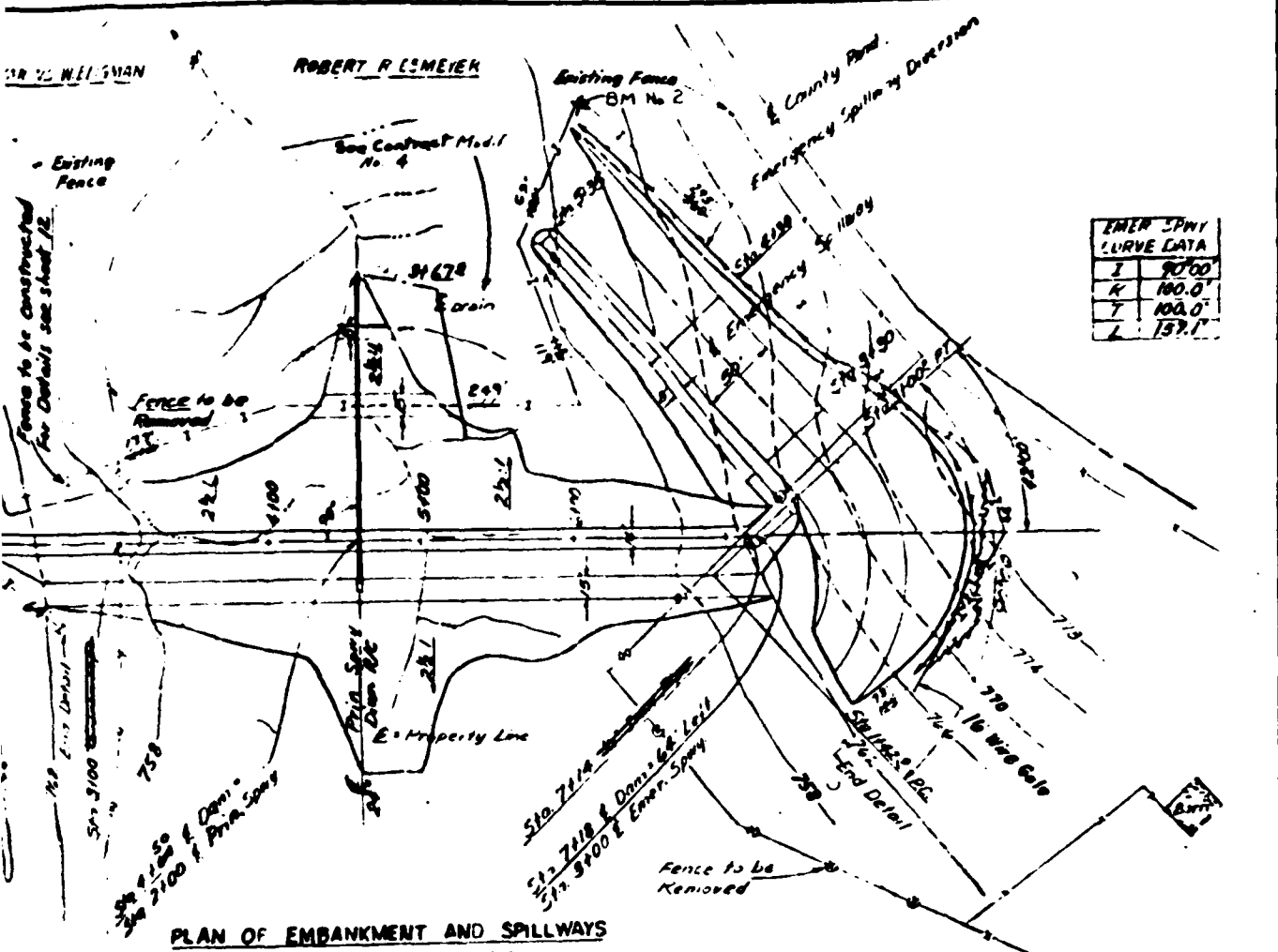


BM No. 2 ELEV. 760.05  
Top of bolt in center of bronze plate  
Stamped BM NO. 2 - 1967 - WELL NAP  
WATERSHED, located 1.5' from Corner Post  
in NE fence corner on Outlet end of emergency  
Sp. Hwy. of Sir. A-21 NE 1/4 Sec. 21, T30N, R20W



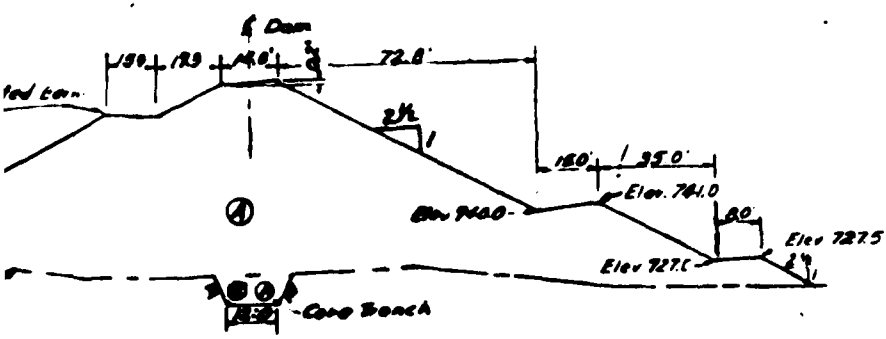
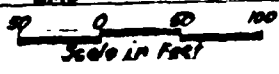
DR. W. WELLSMAN

ROBERT R. LEMEYER



EMER SPWY CURVE DATA	
I	90.00
R	100.0
T	100.0
L	159.1

PLAN OF EMBANKMENT AND SPILLWAYS



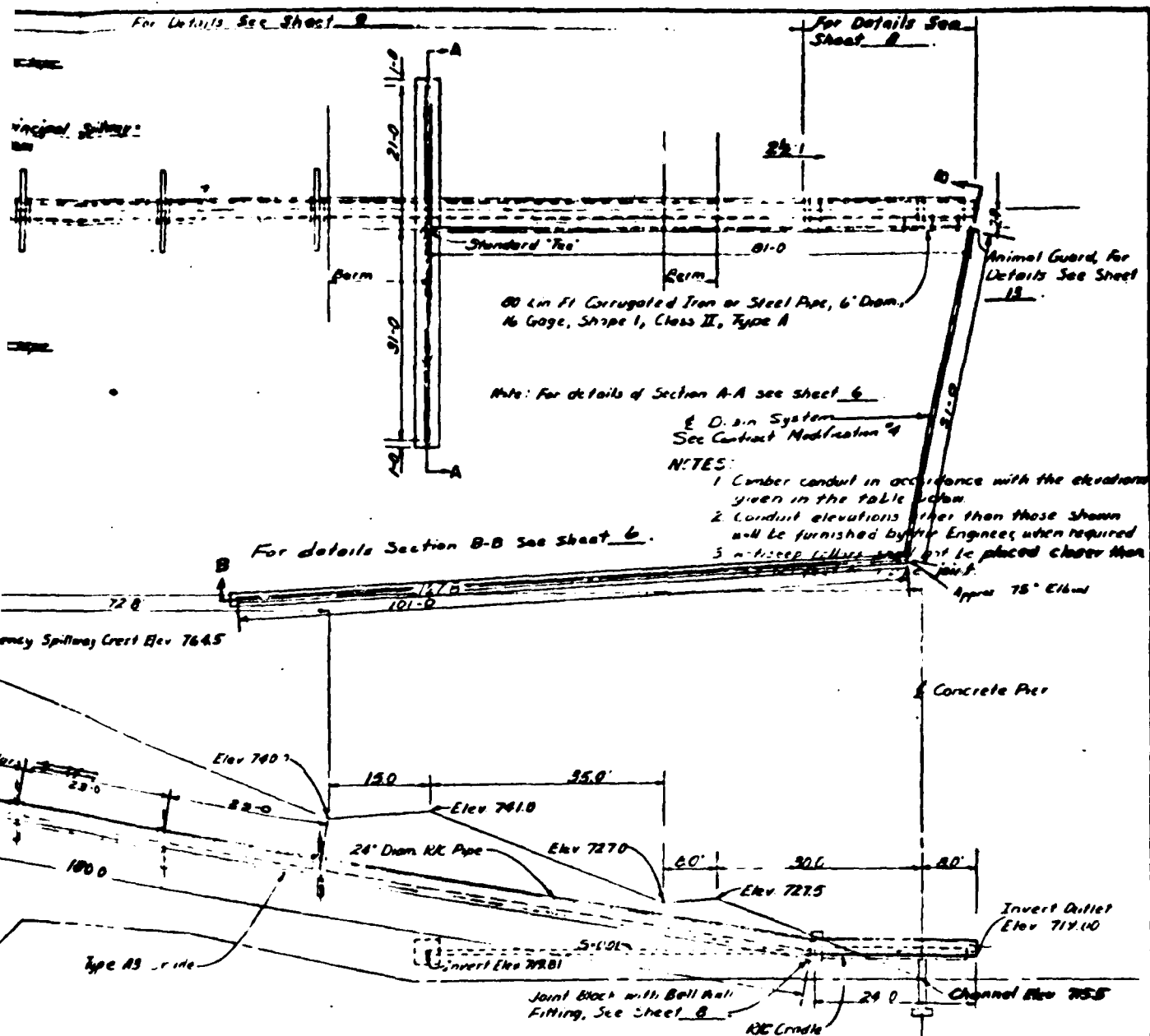
TYPICAL SECTION OF EMBANKMENT

QUANTITIES	
Excavation Common	206
Core French	2298
Total Foundation	2504
Earth Fill	49757
Class A	49757
Fence - Heavy Wire	1244 Lbs

AS BUILT

STRUCTURE A-21	
WELLINGTON-NAPOLEON INTENSIFIED PL 566	
LAFAYETTE COUNTY, MISSOURI	
U. S. DEPARTMENT OF AGRICULTURE	
SOIL CONSERVATION SERVICE	
Drawn by: E. P. Smith	12/50
Checked by: E. P. Smith	12/50
Project No. 56-23,179	





**AS BUILT**

STRUCTURE A-21

R/C DROP INLET FOR 24" DIAM PIPE  
GENERAL LAYOUT  
WELLSFORD-WARREN WATERSHED PL 555  
LAFAYETTE COUNTY, MISSOURI

U. S. DEPARTMENT OF AGRICULTURE  
SOIL CONSERVATION SERVICE

Designed by H. M. Brundage	Date 12/82	Approved by	
Drawn by D. E. Smith	1/85	Checked by	
Field Notes by D. D. Moore	1/85	Field Notes by	
		Project No.	5, E-28, 179

# QUANTITIES

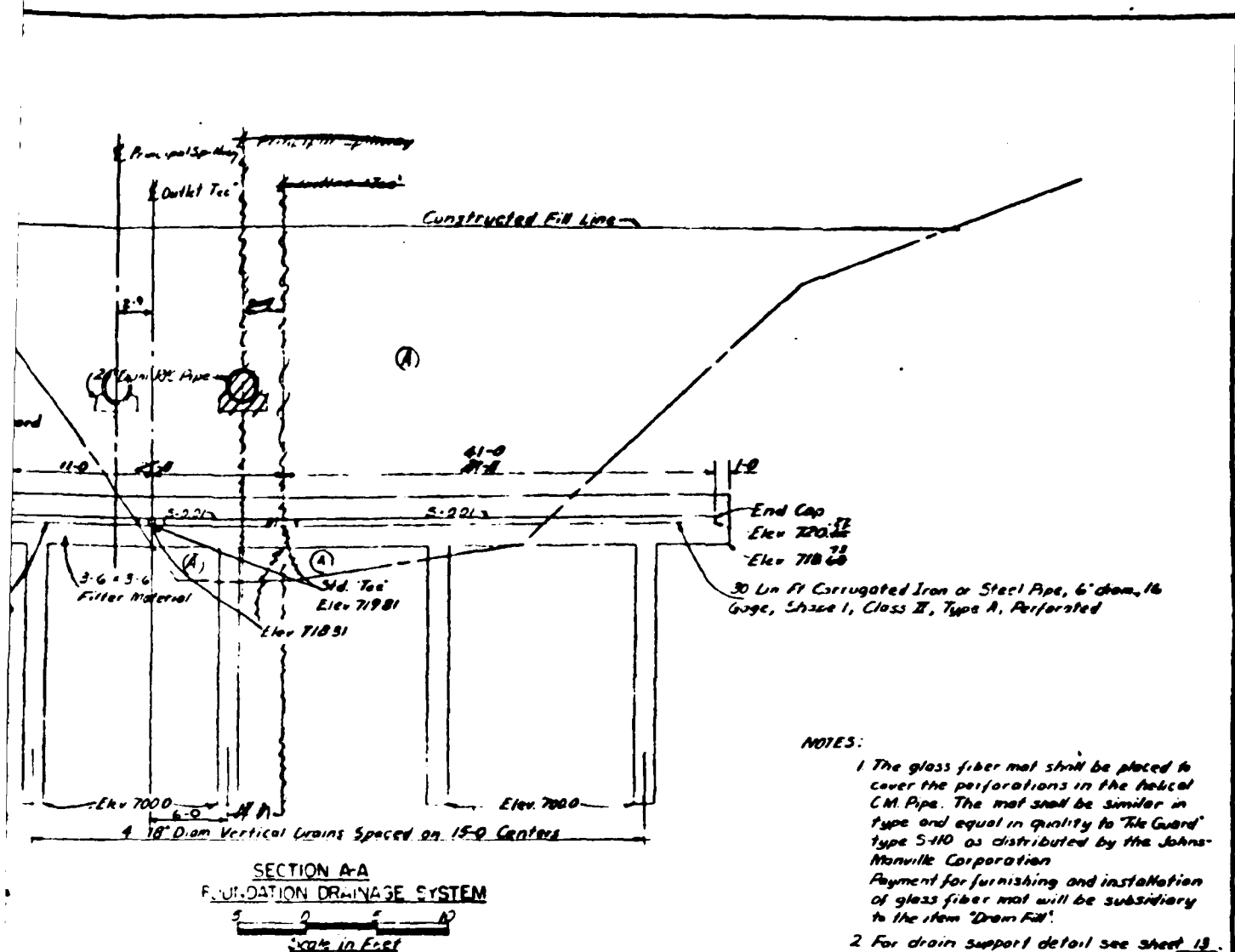
24" Diam, Steel Ring Type Joint and Rubber Gasket	213 Cu Yds
Shape I, Class II, Type A, Perforated	262 Cu Yds
Shape I, Class II, Type A	2541 Mums
	264 Lin Ft
	Lump Sum
	50 Lin Ft
	80 Lin Ft
	30 Cu Yds
	100 Lin Ft
	50 Lin Ft
	9.5 Cu Yds

**SHEET 5 APPENDIX A**

NO 14708 10-65







ing perforated pipe  
1 Sent conforming to  
an at right.

Shape I, Class II, Type A,

FILTER GRADATION	
SEIVE SIZE	PERCENT PASSING
3/8 in.	100
No. 4	85-100
No. 8	80-100
No. 16	50-85
No. 30	25-60
No. 50	10-30
No. 100	5-10
No. 200	Less Than 5

**AS BUILT**

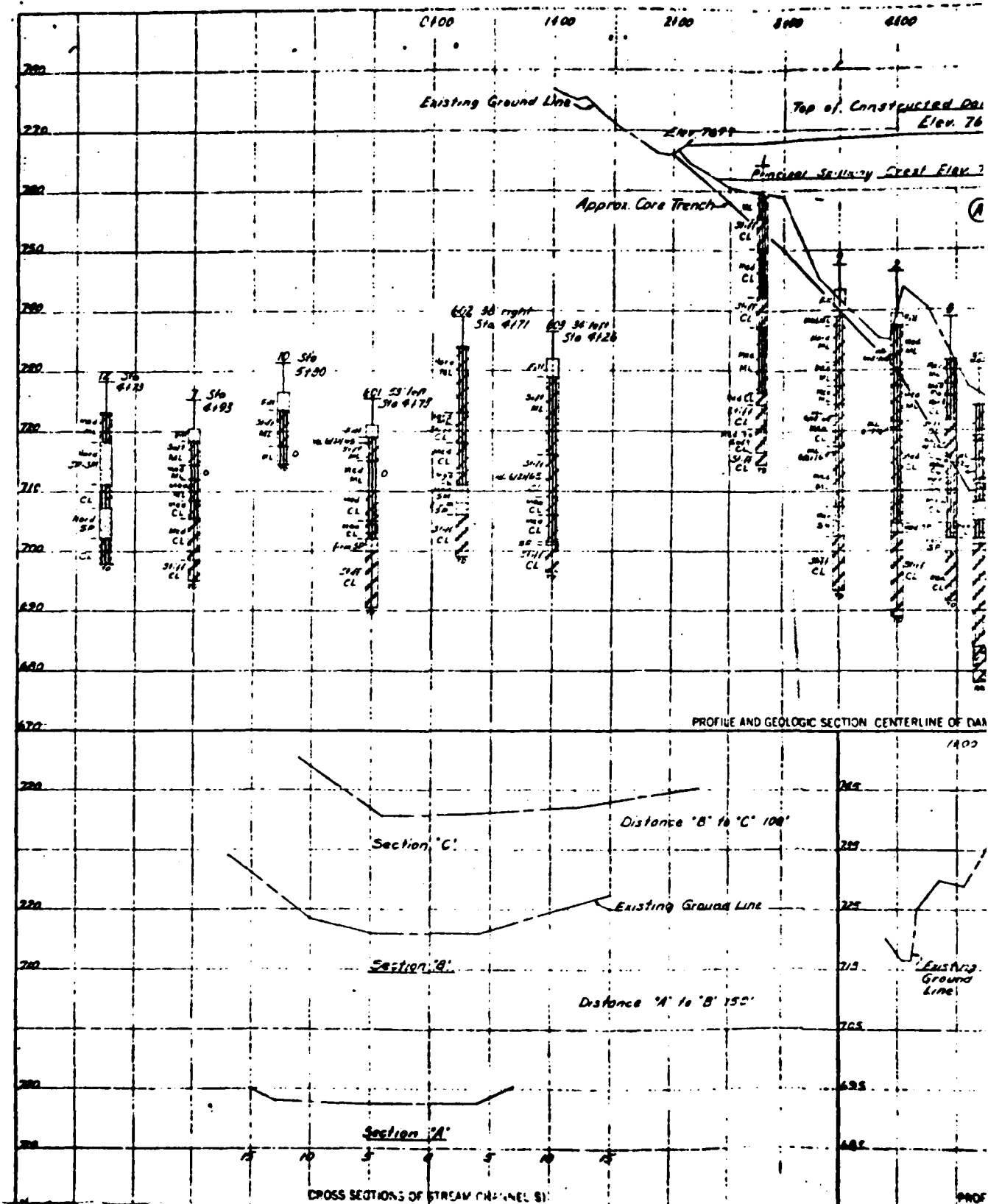
STRUCTURE A-21 FOUNDATION DRAINAGE SYSTEM WELLINGTON-NAPOLEON WATERSHED PL. 566 LAFAYETTE COUNTY, MISSOURI	
U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	
By: H. M. Rasmussen	DATE: 1/66
By: B. E. Smith	DATE: 1/66
By: H. M. Rasmussen	DATE: 2/66
SHEET 6 APPENDIX A	

**SHEET 6 APPENDIX A**

APPENDIX B









10-59

## DETAILED GEOLOGIC INVESTIGATION OF DAM SITES

## GENERAL

State Missouri County Lafayette ; S. 1 T. 25 N. R. 25 W. ; Watershed Hollinsworth  
Subwatershed \_\_\_\_\_ Fund class FP-2-10 (FP-2, WP-1, etc.) Site number A-21 Site group I Structure class A  
Investigated by Huel F. Edwards (signature and title) Equipment used 1951C-10 (Type, size, make, model, etc.) Date 5-15-65

## SITE DATA

Drainage area size 57 sq. mi., 425 acres. Type of structure DT DC Purpose Stabilization  
Direction of valley trend (downstream) NE Maximum height of fill 50.0 feet. Length of fill 565 feet.  
Estimated volume of compacted fill required 40,000 yards

## STORAGE ALLOCATION

	Volume (ac. ft.)	Surface Area (acres)	Depth at Dam (feet)
Sediment	<u>343</u>	<u>20.0</u>	<u>45.0</u>
Floodwater	<u>377</u>	<u>22.7</u>	<u>47.0</u>

## SURFACE GEOLOGY AND PHYSIOGRAPHY

Physiographic description Missouri Basin Loess Topography Holling Altitude of beds: Dip \_\_\_\_\_ Strike \_\_\_\_\_  
Steepness of abutments: Left 25 percent; Right 30 percent. Width of floodplain at centerline of dam 0 feet  
General geology of site: The site is located in moderately rolling land with slopes ranging from 4 to 10 percent. ridge deposits of loess range in depth from approximately 25 to 5 feet in depth and are overlying alluvium, position or bedrock. Bedrock is the Permian, or Pennsylvanian Series of the Pennsylvania System and consist of cyclic deposits and characterized by thin sandstone with thin coal seams of local occurrence. The drainage pattern is well developed.



459

# DETAILED GEOLOGIC INVESTIGATION OF DAM SITES

Centerline of Dam, Principal Spillway, Emergency Spillway, The Stream Channel,

(Centerline of Dam, Principal Spillway, Emergency Spillway, The Stream Channel, Investigations for Design of Structure, Borrow Area, Reservoir Basin, etc.)

## DRILLING PROGRAM

Equipment Used	Number of Holes		Undisturbed (state type)	Number of Samples Taken	
	Exploration	Sampling		Large	Small
FA, SP, T	11	9	8 Shelby	6	16
HA		1		1	
Total	11	10	8	7	16

## SUMMARY OF FINDINGS (Include only factual data)

The loess soil on the abutments is developed to a depth of approximately 10 to 12 feet and classified a medium CL. The underlying loess is classified as medium "L" to low CL. The loess is underlain with alluvial deposits which vary in composition and consistency. The first zone is mostly CL material with blow counts ranging from 5 to 10. This is underlain by a stiff CL with blow counts of 10 or more. Refusal in TH #301 and #6 was at elevation 672 and 675 respectively. Gravelly clay classified CL occurs above the bedrock. Strata, pockets and lens of S, SP and CL with thin stratum of sand were found throughout the central part embankment foundation and the foundation of the principal spillway. Material classified as organic "L" and "H" high in organic matter occurs below the channel, in the right abutment and along the CL of the Principal Spillway.

Borrow areas located are to extend 1000 feet from the centerline of dam.

Borrow Area	Top Soil	Estimated Cubic Yards of Corrected Fill Available
101	5,625	16,275
103	2,000	5,500
Emergency Spillway	250	750

DETAILED GEOLOGIC INVESTIGATION OF DAM SITES

1-59

Wellington-  
State Missouri County Lafayette Watershed Napoleon Subwatershed \_\_\_\_\_  
Number A-21 Site group I Structure class A Investigated by Paul F. Edwards Date 6-15-65  
(signature and title)

INTERPRETATIONS AND CONCLUSIONS

The loess in the abutments below the developed soil profile is classified as medium PL to low CL except for the stiff CL in TH #1 from 17 to 22 feet. Estimated R/C of the medium loess is 4 to 6 and the stiff CL R/C of 10. The material on the right classified as organic PL and PL with organic matter is dark in color and partially decomposed organic material is evident. The R/C was 4 to 10 however the water level was below this zone at the end of drilling but became static at 8.5 feet. The zone was separated in the field on the basis of apparent organic content, however, shelsys were taken in representative areas for laboratory analysis. The underlying CL material had R/C of 5.7, and 9 in TH #3 and from 5 to 13 in TH #6. There was no recovery of split tube samples in TH #6 on some of the drives and blow counts of there are probably not valid. The sand pocket encountered in TH #12, CL station 4 + 75, at five feet was reported to auger hard but appeared loose, wet, and permeable and classified as SM or SP. This pocket was not in TH #301 located at CL station 4 + 69 or TH #601 located 53' down stream from CL station 4 + 73 or TH #602 located 38' upstream from CL station 4 + 71 or TH #603 located 34' downstream from CL station 4 + 26. The greenish colored sand encountered in TH #12 and classified SP at elevation 707 occurs consistently in the central part of the foundation and along the CL of the principal spillway. It occurred at elevation 700 in TH #301 which was 3 feet thick and was one foot thick from elevation 701 to 702. It is 2.5 thick at elevation 702.5 in TH #3, 1 1/2 feet thick at elevation 704.5 in TH #2, 5 feet thick, classified SM, at elevation 707.5 in TH #2. It was found at generally the same elevations in TH #601, 602, 603, and 303. Similar material classified SP was 7 feet thick at elevation 699.6 in TH #303. The gravelly clay classified CL occurring above bedrock in TH #6, #301, and 303 is not permeable.

The emergency spillway will be in loess and highly erosive. Excavations from the spillway used in the embankment will be similar and should be placed like borrow sample #103.1.

Borrow material is scarce. An estimated 10,000 cubic yard will need to be located either within the gully banks or above the crest elevation of the principal spillway. An additional 7000 cubic yards can be obtained by extending the borrow areas to the crest elevation of the emergency spillway. Top soil is normally deep on this soil area ranging up to 2 feet thick. Stone wash covers the top soil on the flatter slopes adjacent to the timber line in the gully banks to an additional depth of approximately 2 feet. The slopes of borrow areas cut steeper than 3 : 1 will increase the hazard of sloughing and slipping on the slopes particularly with in the reservoir area during the life required for the pool to fill.

The area between the gully banks in the foundation is wooded. The root zone will be deep, estimated to be as much as 15'.

The channel is active and cutting. Minimum treatment should be required.

UNITED STATES GOVERNMENT

# Memorandum

TO : W. S. Culpepper, State Conservation Engineer, SCS, Columbia, Missouri 65202

DATE: October 11, 1965

FROM : Rey S. Decker, Head, Soil Mechanics Laboratory, SCS, Lincoln, Nebraska 68508

SUBJECT: ENG 22-5, Missouri WP-08, Wellington-Napoleon, Site No. A-21 (Lafayette County)

## ATTACHMENTS

1. Form SCS-354, Soil Mechanics Laboratory Test Data, 4 sheets.
2. Form SCS-128 and SCS-128A, Consolidation Data, 1 test, 3 sheets.
3. Form SCS-127, Soil Permeability, 1 sheet.
4. Form SCS-355, Triaxial Shear Test Data, 6 sheets.
5. Form SCS-152, Compaction and Penetration Resistance, 7 sheets.
6. Form SCS-357, Summary - Slope Stability Analysis, 2 sheets.
7. Form SCS-372, Recommended Use of Excavated Material, 1 sheet.

## INTERPRETATION AND DISCUSSION OF DATA

### FOUNDATION MATERIALS

- A. Description and Classification. The complicated geology of this site had been quite well interpreted in the report. The site has loess over preconsolidated alluviums with sands that show some evidence of past channeling. The creek channel is degrading at present but organic matter may indicate intermediate reworking.

The loess is logged as CL over soft ML. The alluvium samples which were submitted class as SP-SM, SM, ML, CL, and CH. These materials are stratified and contain pockets of organic material.

Water table is near the base of the loess in most places.

- B. Dry Unit Weight (Blow Count). Six undisturbed samples were submitted. One was a SM on which no density tests were made and the rest varied from ML to CH. The range in dry unit weight for the specimens tested was from 1.04 g/cc to 1.39 g/cc with a corresponding blow count range of 5 to 10 blows per foot. The total range in blow count was from 3 to 13 blows per foot.
- C. Consolidation: A test was made on a specimen from Sample 66W656 at an initial density of 1.53 g/cc as compared to the core opening density of 1.36 g/cc. The test results indicate a consolidation potential of less than .04 ft./ft. under the load of the proposed fill. Since the average density appears to be lower, a potential of .04 ft./ft. will be assumed for a 20-foot depth under the channel area.

2 -- W. S. Culpepper -- 10/11/65

Rey S. Decker

Subj: ENG 22-5, Missouri WP-08, Wellington-Napoleon, Site No. A-21  
(Lafayette County)

At Station 4+73 (conduit location) the blow count and materials logged indicate only about half this potential, or 0.6 foot in a 26-foot depth. Based on  $b = 260$  feet,  $h = 42$  feet,  $d = 26$  feet, a maximum horizontal strain of only .004 ft./ft. is computed.

- D. Permeability: Both horizontal and vertical tests were made on the SM sample. Rates of  $k_h = 0.2$  ft./day and  $k_v = .006$  ft./day were reported. During the consolidation test, a rate of  $k = 0.02$  ft./day was found for the ML specimen.

Rates as high as  $k = 20$  ft./day are indicated for the clean SP-SM submitted from TH No. 12. Unless it can be proven to be a very limited pocket, not extending upstream or downstream beyond the toes of the fill, it will be necessary to cut off or drain this stratum represented by Sample 66W661 (12.3) and indicated in the logs as extending from 16 feet to 21 feet in depth.

- E. Shear Strength: The following shear tests were performed:

Sample Number	Location	Material	Test	Shear Strength
66W656	Hole No. 6, 15.5'-17'	CL	Triaxial CU*	27.5° - 425
66W657	Hole No. 6, 30.5'-32'	CH	$q_u$	0° - 2550
66W658	Hole No. 10, 10.5'-11'	ML	Triaxial CU*	18.5° - 1050
66W659	Hole No. 11, 20.5'-22'	CL	$q_u$	0° - 1400

\*CU - Consolidated, undrained.

The weaker shear strength indicated by the test on 66W656 may extend down as deep as 20 feet, based on blow count, in the channel area only.

#### EMBANKMENT MATERIALS

- A. Classification: Borrow samples submitted all class as CL and are all fine materials with LL varying from 36 to 42 and PI from 13 to 20.
- B. Compacted Dry Density: Standard Proctor tests were made on all the large-bag samples. Maximum dry densities varied from 100.5 p.c.f. to 104.5 p.c.f.
- C. Permeability: All compacted borrow like the samples submitted will be low in permeability.

3 -- W. S. Culpepper -- 10/11/65

Rey S. Decker

Subj: ENG 22-5, Missouri WP-08, Wellington-Napoleon, Site No. A-21  
(Lafayette County)

- D. Shear Strength: Consolidated, undrained triaxial shear tests were made on two samples. The tests were made on specimens molded to 95% of standard density and soaked before shearing. Test values were  $\phi = 11.5^\circ$ ,  $c = 925$  p.s.f. for 66W662 and  $\phi = 19^\circ$ ,  $c = 850$  p.s.f. for 66W663.
- E. Consolidation: No tests were made. Based on the nature of these materials and previous tests, a residual settlement of 2.5% of the fill height at the channel is expected.

#### SLOPE STABILITY ANALYSIS

Slope stability was checked by a Swedish circle method. A phreatic line was assumed as developed from the emergency spillway crest and impinging on the downstream slope at the berm, elevation 740. Cracking was assumed down to the phreatic line. Weak foundation was assumed to a 22-foot depth.

Even with these severe assumptions, a minimum upstream safety factor of 1.29 against full drawdown and a downstream safety factor of 1.58 were computed.

#### SETTLEMENT STRAINS

Differential settlement will cause some strains but the plastic embankment materials should be able to adjust without cracking.

#### CONCLUSIONS AND RECOMMENDATIONS

- A. Cutoff: It appears impractical to cut off all the organic or stratified material and sand pockets noted in this foundation. A cutoff trench of moderate depth (5 feet to 8 feet) is suggested with a short drain.

- B. Principal Spillway: The proposed location has foundation conditions acceptable for a concrete pipe.

Use a pipe camber of 0.5 foot.

Base pipe joint design on a maximum horizontal strain of .004 ft./ft.

- C. Drainage: Due to the stratification, organic materials and sand pockets, it is believed desirable to provide drainage under the downstream berm from about  $\frac{1}{4}$  Station 4+50 to 5+50. The drain trench should be 8 feet to 10 feet deep to contact the more permeable strata and outlet into a slotted pipe.

Unless it is proven that the deep sand pocket represented by Sample 66W661 (12.3) does not extend upstream or downstream beyond the toes of the proposed fill, it must be relieved by a well. A deep pit backfilled with filter sand would accomplish this if this would be more economical than a standard well.

4 -- W. S. Culpepper -- 10/11/65

Rey S. Decker

Subj: ENG 22-5, Missouri WP-08, Wellington-Napoleon, Site No. A-21  
(Lafayette County)

Filter sand should be composed of a fine, well-graded sand like ASTM fine concrete aggregate. A coarse segment would then be required around the drain pipes.

- D. Embankment Design: A homogeneous fill of CL materials compacted to 95% of standard is recommended.

Make both slopes 2 1/2:1 with berms of 15 feet at elevation 762 upstream and 740 downstream as proposed.

Provide overfill of 1.5 feet at the maximum section to compensate for residual settlement of about 0.6 foot in the foundation and 0.9 foot in the embankment.

Prepared by:

R. B. Phillips  
Roland B. Phillips

#### Attachments

cc: W. S. Culpepper (1)  
Higg, Project Engineer  
C. A. Reese, Lincoln, Nebraska  
D. S. McVicker, Lincoln, Nebraska



[illegible]





**ORIFICE**

**CONDUIT**

**WEIR**

**CONDUIT**

**FLOOD ROUTING**

**HYDROGRAPH EXTENSION**

**SUMMARY DATA**

**AS BUILT**

**FLOOD ROUTING**

## STRUCTURE DATA

[illegible]

## CHANNEL HYDRAULICS

GENERAL INFORMATION		FINANCIAL DATA		OPERATIONAL DATA	
DATE	TIME	AMOUNT	CURRENCY	QUANTITY	UNIT
1970-01-01	08:00	100.00	USD	100	KG
1970-01-01	09:00	200.00	USD	200	KG
1970-01-01	10:00	300.00	USD	300	KG
1970-01-01	11:00	400.00	USD	400	KG
1970-01-01	12:00	500.00	USD	500	KG
1970-01-01	13:00	600.00	USD	600	KG
1970-01-01	14:00	700.00	USD	700	KG
1970-01-01	15:00	800.00	USD	800	KG
1970-01-01	16:00	900.00	USD	900	KG
1970-01-01	17:00	1000.00	USD	1000	KG
1970-01-01	18:00	1100.00	USD	1100	KG
1970-01-01	19:00	1200.00	USD	1200	KG
1970-01-01	20:00	1300.00	USD	1300	KG
1970-01-01	21:00	1400.00	USD	1400	KG
1970-01-01	22:00	1500.00	USD	1500	KG
1970-01-01	23:00	1600.00	USD	1600	KG
1970-01-02	00:00	1700.00	USD	1700	KG
1970-01-02	01:00	1800.00	USD	1800	KG
1970-01-02	02:00	1900.00	USD	1900	KG
1970-01-02	03:00	2000.00	USD	2000	KG
1970-01-02	04:00	2100.00	USD	2100	KG
1970-01-02	05:00	2200.00	USD	2200	KG
1970-01-02	06:00	2300.00	USD	2300	KG
1970-01-02	07:00	2400.00	USD	2400	KG
1970-01-02	08:00	2500.00	USD	2500	KG
1970-01-02	09:00	2600.00	USD	2600	KG
1970-01-02	10:00	2700.00	USD	2700	KG
1970-01-02	11:00	2800.00	USD	2800	KG
1970-01-02	12:00	2900.00	USD	2900	KG
1970-01-02	13:00	3000.00	USD	3000	KG
1970-01-02	14:00	3100.00	USD	3100	KG
1970-01-02	15:00	3200.00	USD	3200	KG
1970-01-02	16:00	3300.00	USD	3300	KG
1970-01-02	17:00	3400.00	USD	3400	KG
1970-01-02	18:00	3500.00	USD	3500	KG
1970-01-02	19:00	3600.00	USD	3600	KG
1970-01-02	20:00	3700.00	USD	3700	KG
1970-01-02	21:00	3800.00	USD	3800	KG
1970-01-02	22:00	3900.00	USD	3900	KG
1970-01-02	23:00	4000.00	USD	4000	KG
1970-01-03	00:00	4100.00	USD	4100	KG
1970-01-03	01:00	4200.00	USD	4200	KG
1970-01-03	02:00	4300.00	USD	4300	KG
1970-01-03	03:00	4400.00	USD	4400	KG
1970-01-03	04:00	4500.00	USD	4500	KG
1970-01-03	05:00	4600.00	USD	4600	KG
1970-01-03	06:00	4700.00	USD	4700	KG
1970-01-03	07:00	4800.00	USD	4800	KG
1970-01-03	08:00	4900.00	USD	4900	KG
1970-01-03	09:00	5000.00	USD	5000	KG
1970-01-03	10:00	5100.00	USD	5100	KG
1970-01-03	11:00	5200.00	USD	5200	KG
1970-01-03	12:00	5300.00	USD	5300	KG
1970-01-03	13:00	5400.00	USD	5400	KG
1970-01-03	14:00	5500.00	USD	5500	KG
1970-01-03	15:00	5600.00	USD	5600	KG
1970-01-03	16:00	5700.00	USD	5700	KG
1970-01-03	17:00	5800.00	USD	5800	KG
1970-01-03	18:00	5900.00	USD	5900	KG
1970-01-03	19:00	6000.00	USD	6000	KG
1970-01-03	20:00	6100.00	USD	6100	KG
1970-01-03	21:00	6200.00	USD	6200	KG
1970-01-03	22:00	6300.00	USD	6300	KG
1970-01-03	23:00	6400.00	USD	6400	KG
1970-01-04	00:00	6500.00	USD	6500	KG
1970-01-04	01:00	6600.00	USD	6600	KG
1970-01-04	02:00	6700.00	USD	6700	KG
1970-01-04	03:00	6800.00	USD	6800	KG
1970-01-04	04:00	6900.00	USD	6900	KG
1970-01-04	05:00	7000.00	USD	7000	KG
1970-01-04	06:00	7100.00	USD	7100	KG
1970-01-04	07:00	7200.00	USD	7200	KG
1970-01-04	08:00	7300.00	USD	7300	KG
1970-01-04	09:00	7400.00	USD	7400	KG
1970-01-04	10:00	7500.00	USD	7500	KG
1970-01-04	11:00	7600.00	USD	7600	KG
1970-01-04	12:00	7700.00	USD	7700	KG
1970-01-04	13:00	7800.00	USD	7800	KG
1970-01-04	14:00	7900.00	USD	7900	KG
1970-01-04	15:00	8000.00	USD	8000	KG
1970-01-04	16:00	8100.00	USD	8100	KG
1970-01-04	17:00	8200.00	USD	8200	KG
1970-01-04	18:00	8300.00	USD	8300	KG
1970-01-04	19:00	8400.00	USD	8400	KG
1970-01-04	20:00	8500.00	USD	8500	KG
1970-01-04	21:00	8600.00	USD	8600	KG
1970-01-04	22:00	8700.00	USD	8700	KG
1970-01-04	23:00	8800.00	USD	8800	KG
1970-01-05	00:00	8900.00	USD	8900	KG
1970-01-05	01:00	9000.00	USD	9000	KG
1970-01-05	02:00	9100.00	USD	9100	KG
1970-01-05	03:00	9200.00	USD	9200	KG
1970-01-05	04:00	9300.00	USD	9300	KG
1970-01-05	05:00	9400.00	USD	9400	KG
1970-01-05	06:00	9500.00	USD	9500	KG
1970-01-05	07:00	9600.00	USD	9600	KG
1970-01-05	08:00	9700.00	USD	9700	KG
1970-01-05	09:00	9800.00	USD	9800	KG
1970-01-05	10:00	9900.00	USD	9900	KG
1970-01-05	11:00	10000.00	USD	10000	KG
1970-01-05	12:00	10100.00	USD	10100	KG
1970-01-05	13:00	10200.00	USD	10200	KG
1970-01-05	14:00	10300.00	USD	10300	KG
1970-01-05	15:00	10400.00	USD	10400	KG
1970-01-05	16:00	10500.00	USD	10500	KG
1970-01-05	17:00	10600.00	USD	10600	KG
1970-01-05	18:00	10700.00	USD	10700	KG
1970-01-05	19:00	10800.00	USD	10800	KG
1970-01-05	20:00	10900.00	USD	10900	KG
1970-01-05	21:00	11000.00	USD	11000	KG
1970-01-05	22:00	11100.00	USD	11100	KG
1970-01-05	23:00	11200.00	USD	11200	KG
1970-01-06	00:00	11300.00	USD	11300	KG
1970-01-06	01:00	11400.00	USD	11400	KG
1970-01-06	02:00	11500.00	USD	11500	KG
1970-01-06	03:00	11600.00	USD	11600	KG
1970-01-06	04:00	11700.00	USD	11700	KG
1970-01-06	05:00	11800.00	USD	11800	KG
1970-01-06	06:00	11900.00	USD	11900	KG
1970-01-06	07:00	12000.00	USD	12000	KG
1970-01-06	08:00	12100.00	USD	12100	KG
1970-01-06	09:00	12200.00	USD	12200	KG
1970-01-06	10:00	12300.00	USD	12300	KG
1970-01-06	11:00	12400.00	USD	12400	KG
1970-01-06	12:00	12500.00	USD	12500	KG
1970-01-06	13:00	12600.00	USD	12600	KG
1970-01-06	14:00	12700.00	USD	12700	KG
1970-01-06	15:00	12800.00	USD	12800	KG
1970-01-06	16:00	12900.00	USD	12900	KG
1970-01-06	17:00	13000.00	USD	13000	KG
1970-01-06	18:00	13100.00	USD	13100	KG
1970-01-06	19:00	13200.00	USD	13200	KG
1970-01-06	20:00	13300.00	USD	13300	KG
1970-01-06	21:00	13400.00	USD	13400	KG
1970-01-06	22:00	13500.00	USD	13500	KG
1970-01-06	23:00	13600.00	USD	13600	KG
1970-01-07	00:00	13700.00	USD	13700	KG
1970-01-07	01:00	13800.00	USD	13800	KG
1970-01-07	02:00	13900.00	USD	13900	KG
1970-01-07	03:00	14000.00	USD	14000	KG
1970-01-07	04:00	14100.00	USD	14100	KG
1970-01-07	05:00	14200.00	USD	14200	KG
1970-01-07	06:00	14300.00	USD	14300	KG
1970-01-07	07:00	14400.00	USD	14400	KG
1970-01-07	08:00	14500.00	USD	14500	KG
1970-01-07	09:00	14600.00	USD	14600	KG
1970-01-07	10:00	14700.00	USD	14700	KG
1970-01-07	11:00	14800.00	USD	14800	KG
1970-01-07	12:00	14900.00	USD	14900	KG
1970-01-07	13:00	15000.00	USD	15000	KG
1970-01-07	14:00	15100.00	USD	15100	KG
1970-01-07	15:00	15200.00	USD	15200	KG
1970-01-07	16:00	15300.00	USD	15300	KG
1970-01-07	17:00	15400.00	USD	15400	KG
1970-01-07	18:00	15500.00	USD	15500	KG
1970-01-07	19:00	15600.00	USD	15600	KG
1970-01-07	20:00	15700.00	USD	15700	KG
1970-01-07	21:00	15800.00	USD	15800	KG
1970-01-07	22:00	15900.00	USD	15900	KG
1970-01-07	23:00	16000.00	USD	16000	KG
1970-01-08	00:00	16100.00	USD	16100	KG
1970-01-08	01:00	16200.00	USD	16200	KG
1970-01-08	02:00	16300.00	USD	16300	KG
1970-01-08	03:00	16400.00	USD	16400	KG
1970-01-08	04:00	16500.00	USD	16500	KG
1970-01-08	05:00	16600.00	USD	16600	KG
1970-01-08	06:00	16700.00	USD	16700	KG
1970-01-08	07:00	16800.00	USD	16800	KG
1970-01-08	08:00	16900.00	USD	16900	KG
1970-01-08	09:00	17000.00	USD	17000	KG
1970-01-08	10:00	17100.00	USD	17100	KG
1970-01-08	11:00	17200.00	USD	17200	KG
1970-01-08	12:00	17300.00	USD	17300	KG
1970-01-08	13:00	17400.00	USD	17400	KG
1970-01-08	14:00	17500.00	USD	17500	KG
1970-01-08	15:00	17600.00	USD	17600	KG
1970-01-08	16:00	17700.00	USD	17700	KG
1970-01-08	17:00	17800.00	USD	17800	KG
1970-01-08	18:00	17900.00	USD	17900	KG
1970-01-08	19:00	18000.00	USD	18000	KG
1970-01-08	20:00	18100.00	USD	18100	KG
1970-01-08	21:00	18200.00	USD	18200	KG
1970-01-08	22:00	18300.00	USD	18300	KG
1970-01-08	23:00	18400.00	USD	18400	KG
1970-01-09	00:00	18500.00	USD	18500	KG
1970-01-09	01:00	18600.00	USD	18600	KG
1970-01-09	02:00	18700.00	USD	18700	KG
1970-01-09	03:00	18800.00	USD	18800	KG
1970-01-09	04:00	18900.00	USD	18900	KG
1970-01-09	05:00	19000.00	USD	19000	KG
1970-01-09	06:00	19100.00	USD	19100	KG
1970-01-09	07:00	19200.00	USD	19200	KG
1970-01-09	08:00	19300.00	USD	19300	KG
1970-01-09	09:00	19400.00	USD	19400	KG
1970-01-09	10:00	19500.00	USD	19500	KG
1970-01-09	11:00	19600.00			

## STRUCTURE DATA

STRUCTURE	SITE NUMBER			A 21
	TIME OF CONCENTRATION	HOURS		0.327
	RUNOFF COEFFICIENT			70
INFLOW	MOISTURE CONDITION			II
	STORM DISTRIBUTION CURVE			II
HYDROGRAPH	HYDROGRAPH FAMILY			II
	TO/VS USED			10
DESIGN	STORM DURATION	FEET		25
	STORM DURATION	HOURS		10
DATA	POINT RAINFALL	FEET		5.07
	POINT RAINFALL	INCHES		0.25
	POINT RAINFALL	FEET		11.25
	POINT RAINFALL	INCHES		2.25
	POINT RAINFALL	FEET		7.50
	STORM FREQ FOR SPILLWAY	YEARS		0.5
	STORM FREQ FOR SPILLWAY	YEARS		2
	STORM FREQ FOR SPILLWAY	FEET		1.1
EMERGENCY	STORM FREQ FOR SPILLWAY	CFS/FT		2.50
	STORM FREQ FOR SPILLWAY	FT/SEC		4.8
SPILLWAY	STORM FREQ FOR SPILLWAY	FT/FT		0.039
	STORM FREQ FOR SPILLWAY	FT/SEC		9.0
DATA	STORM FREQ FOR SPILLWAY	FT/FT		361
	STORM FREQ FOR SPILLWAY	FT/SEC		4.7
	STORM FREQ FOR SPILLWAY	HOURS		15.1
STRUCTURE	HEIGHT & STORAGE			19.837
	CLASS			4

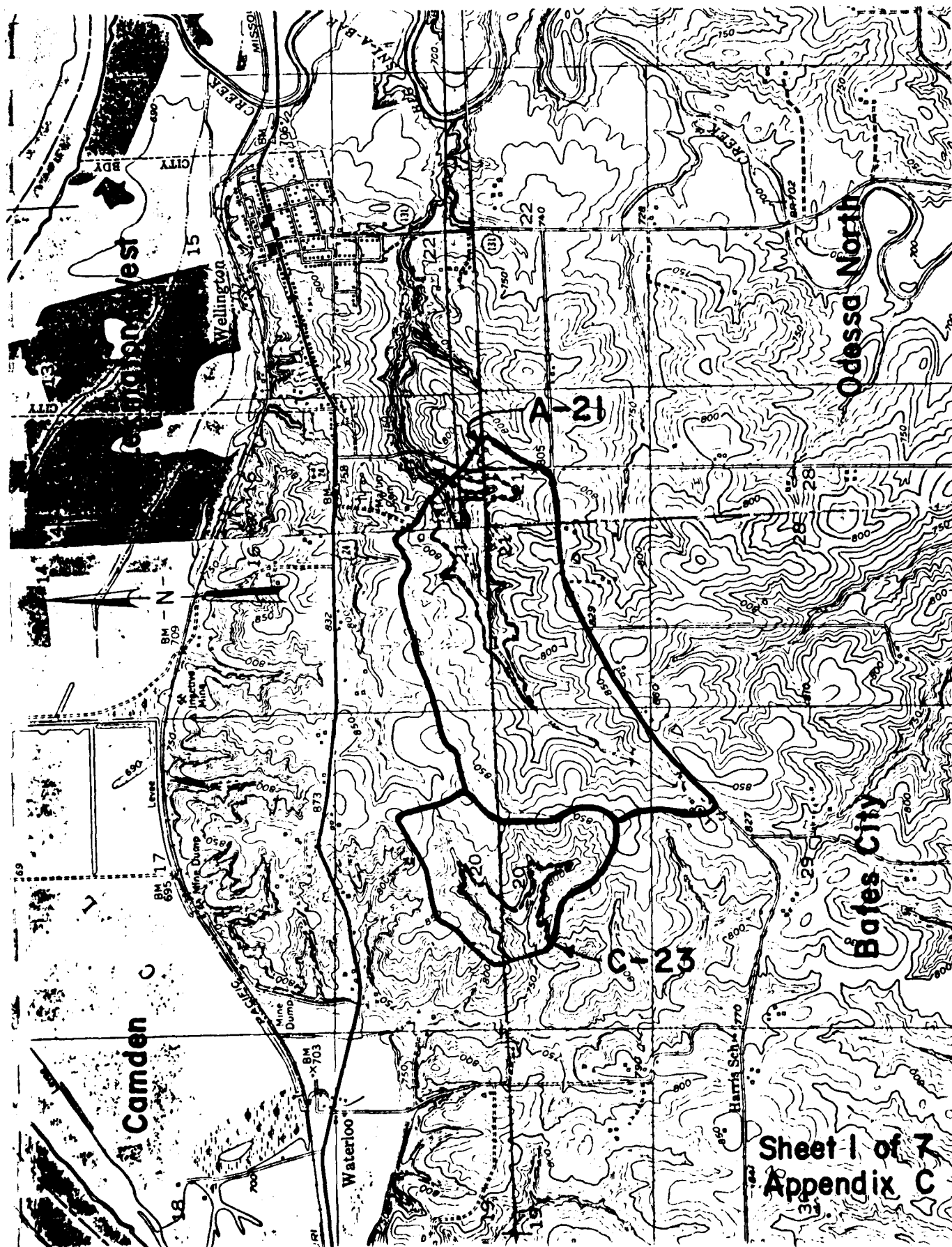
## CHANNEL HYDRAULICS

AS BUILT

GENERAL DESIGN DATA  
 STRUCTURE A-21  
 WELLINGTON-NAPOLEON WATERSHED PL566  
 LAFAYETTE COUNTY, MISSOURI  
 U S DEPARTMENT OF AGRICULTURE  
 SOIL CONSERVATION SERVICE

Drawn by: Mon & Randall 1965  
 Smith V66  
 No. 11 V66  
 No. 2 52-23,179-H

APPENDIX C



## HYDRAULIC AND HYDROLOGIC DATA

### DESIGN DATA From As-Built Plans and Field Measurements

EXPERIENCE DATA: No records are available. Owner stated that to his knowledge the lake normally remains at pool level elevation with some overflow. The apparent high water mark is at elevation 764.0 ft which is about 0.2 ft below the emergency spillway crest of 764.2.

VISUAL INSPECTION: At the time of inspection, the reservoir pool elevation was 761.7, which is about 0.2 ft above the measured crest elevation of 761.5.

OVERTOPPING POTENTIAL: Flood routings were performed to determine the overtopping potential. Since the dam is of intermediate size with a high hazard rating, a Spillway Design Storm of 100 percent of the PMF was prescribed by the guidelines. The Probable Maximum Flood (PMF) is defined as the flood discharge that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. Reservoir area and storage data and the watershed drainage data were obtained from the As-Built plans. A 5 minute interval unit graph was developed for this watershed area which resulted in a peak inflow of 1335 c.f.s. and a time to peak of 15 minutes. Application of the probable maximum precipitation minus losses resulted in a flood hydrograph peak inflow of 7453 c.f.s. (see Sheet 5 of 7). Rainfall distribution for the 24 hour storm was according to EM 1110-2-1411. Considering all factors, the combination of dam, spillway and storage is not sufficient to pass the PMF without overtopping. The embankment crest (El. 768.0) would be overtopped by 1.99 ft at flood pool elevation 769.99.

Fifty percent of the PMF was routed and overtopped the dam by .83 ft through the spillways. The portion of the PMF that will just reach the top of the dam at elevation 768.0 ft is about 0.33. This flood event is in excess of a 100 year frequency flood. For additional data see Summary of Dam Safety Analysis, Sheets 3 and 4 of this Appendix.

## OVERTOPPING ANALYSIS FOR Dam A-21

### INPUT PARAMETERS

1. Unit Hydrograph - SCS Dimensionless - Flood Hydrograph Package (HEC-1); Dam Safety Version Was Used.  
Hydraulic Inputs Are As Follows:
  - a. Twenty-four Hour Rainfall of 25 Inches  
For 200 Square Miles - All Season Envelope
  - b. Drainage Area = 426 Acres; = .67 Sq. Miles
  - c. Travel Time of Runoff 0.33 Hrs.; Lag Time 0.2 Hrs.
  - d. Soil Conservation Service Runoff Curve No. 85 (AMC III)
  - e. Proportion of Drainage Basin Impervious 0.05
2. Spillways
  - a. Rating Curve for Primary Spillway: Drop Inlet Concrete Structure (Crest El. 761.5) with 24 in. diameter RCP Pipe
  - b. Emergency Spillway: Trapezoidal Cut-Seeded (Crest El. 764.2)  
Length 50 Ft.; Side Slopes 3:1; C = 2.65
  - c. Dam Overflow  
Length 520 Ft.; Side Slopes vertical; C = 3.0

Note: Combined Spillway and Dam Rating curve computed by Hanson Engineers. Data Provided To Computer on Y4 and Y5 Cards.

### SUMMARY OF DAM SAFETY ANALYSIS

1. Unit Hydrograph
  - a. Peak - 1335 c.f.s.
  - b. Time to Peak 15 Min.
2. Flood Routings Were Computed by the Modified Puls Method
  - a. Peak Inflow (see Sheet 5)  
50% PMF 3726 c.f.s.; 100% PMF 7453 c.f.s.



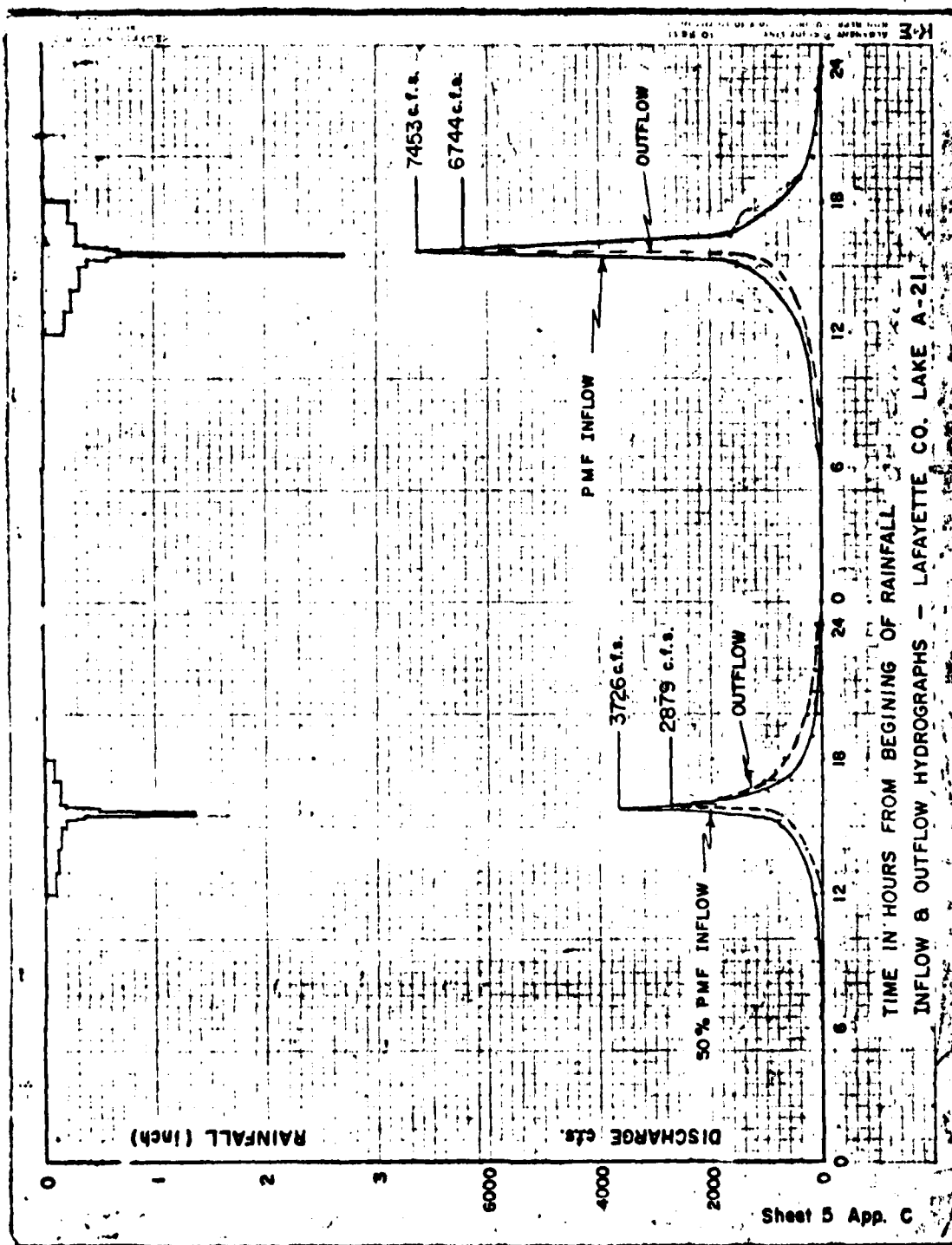
b. Maximum Reservoir Elevation

50% PMF 768.83 100% PMF 769.99

c. Portion of PMF That Will Reach Top of Dam

33 %; Top of Dam Elev. 768.0 Ft.

3. Computer Input and Output Data Sheets 6 and 7



## LAFAYETTE CO DAY A-21 PROBABLE MAXIMUM FLOOD (INPUT DATA)

A \*\*\*\*\* OVERTOPPING ANALYSIS FOR LAFAYETTE CO. DAM A-21 (HEC-1) DAM SAFETY PROGRAM  
 A \*\*\*\*\* CO. CODE 107 CO NAME LAFAYETTE STATE ID NO. NO. 10144 OWR WELLINGTON-NAPO  
 A \*\*\*\*\* HANSON ENGINEERS INC DAM SAFETY INSPECTION (JOB NO. 03778) \*\*\*\*\*

B	300	0	5	0	0	0	0	0	4	0
---	-----	---	---	---	---	---	---	---	---	---

81 5

↓ 1 7 1

0.1	0.30	0.35	0.40	0.45	0.50	1.00
-----	------	------	------	------	------	------

K	0	1	3	1
---	---	---	---	---

1 \*\*\*\*\* INFLOW HYDROGRAPH COMPUTATION \*\*\*\*\*

1	2	0.67	1.00
---	---	------	------

25.0	102	120	130
------	-----	-----	-----

-1 -85 05

~~W2 0.33 20~~

X	0.0	-10	2.0
---	-----	-----	-----

K	1	A21	0	1
---	---	-----	---	---

+ \*\*\*\*\* RESERVOIR ROUTING BY MODIFIED PULS \*\*\*\*\*+

Y

Y1 1 343 -1

761	762	763	764	765	766	767	768	769	770	771
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----

95	0	40	80	128	345	682	1130	3246	6765	11238
----	---	----	----	-----	-----	-----	------	------	------	-------

\$6	0	343	363	386	410	435	460	485	510	535
-----	---	-----	-----	-----	-----	-----	-----	-----	-----	-----

55 560 585

729	762	763	764	765	766	767	768	769	770
-----	-----	-----	-----	-----	-----	-----	-----	-----	-----

\$E	771	772
-----	-----	-----

55 76: 5

11. 19

100

TA)

(HEC-1) DAM SAFETY PROGRAM \*\*\*\*\*  
10144 OWR WELLINGTON-NAPOLEON WATERSHED DISTRICT \*\*\*\*\*  
B NO 03778) \*\*\*\*\*  
0 4 0

-85 .05

-1		
769	770	771
3246	6765	11238
485	510	535
768	769	770

\*\*\*\*\*

\*\*\*\*\* PAGE 0001

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LAFAYETTE CO DAM A-21 PROBABLE MAXIMUM FLOOD (OUTPUT DATA)

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO  
 FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER  
 AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLO			
				RATIO 1 0.10	RATIO 2 0.30	RATIO 3 0.35	RATIO 4 0.40
HYDROGRAPH AT	1	0.67	1	745.	2236.	2609.	2981.
	(	1.74)	(	21.10)(	63.31)(	73.87)(	84.42)(
ROUTED TO	21	0.67	1	114.	1017.	1487.	2017.
	(	1.74)	(	3.22)(	28.80)(	42.11)(	57.11)(

## SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1

	INITIAL VALUE	SPILLWAY CREST
ELEVATION	762.00	761.50
STORAGE	343.	338.
OUTFLOW	13.	0

RATIO OF PMF	MAXIMUM RESERVOIR W.S. ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURAT OVER HOUR
0.10	764.70	0.00	403	114	0.1
0.30	767.75	0.00	479	101	0.1
0.35	768.17	0.17	489	1487	0.3
0.40	768.42	0.42	495	2017	0.5
0.45	768.63	0.63	501	2472	0.6
0.50	768.83	0.83	506	2879	0.8
1.00	769.99	1.99	535	6744	4.0

\*\*\*\*\*

\*\*\*\*\*

\*\*\*\*\*

UT DATA)

SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS  
 ET PER SECOND (CUBIC METERS PER SECOND)  
 UARE MILES (SQUARE KILOMETERS)

RATIOS APPLIED TO FLOWS					
RATIO 2	RATIO 3	RATIO 4	RATIO 5	RATIO 6	RATIO 7
0.30	0.35	0.40	0.45	0.50	1.00
2236.	2609.	2981.	3354.	3726.	7453.
63.31)(	73.87)(	84.42)(	94.97)(	105.52)(	211.04)(
1017.	1487.	2017.	2472.	2879.	6744.
28.80)(	42.11)(	57.11)(	70.00)(	81.52)(	190.97)(

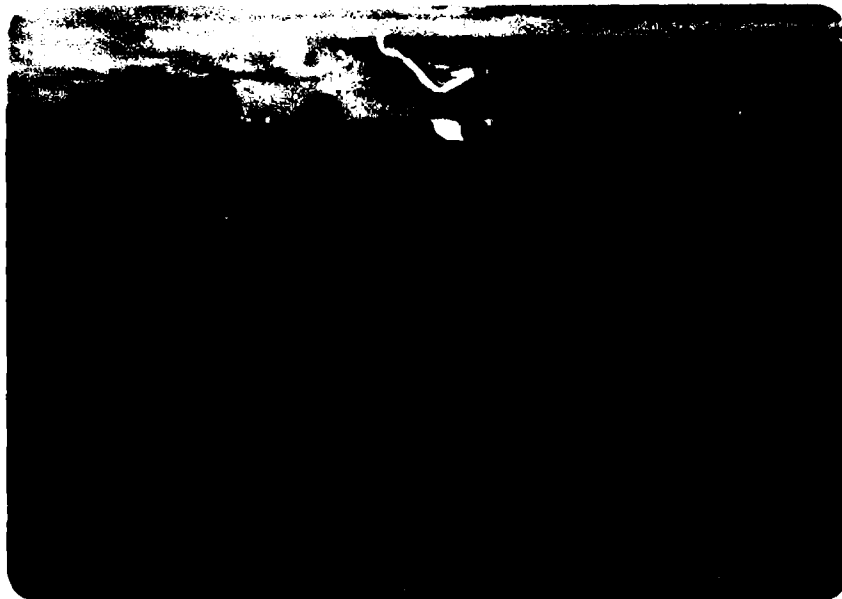
SUMMARY OF DAM SAFETY ANALYSIS

FINAL VALUE	SPILLWAY CREST	TOP OF DAM
762.00	761.50	768.00
343.	338.	485.
13	0	1130

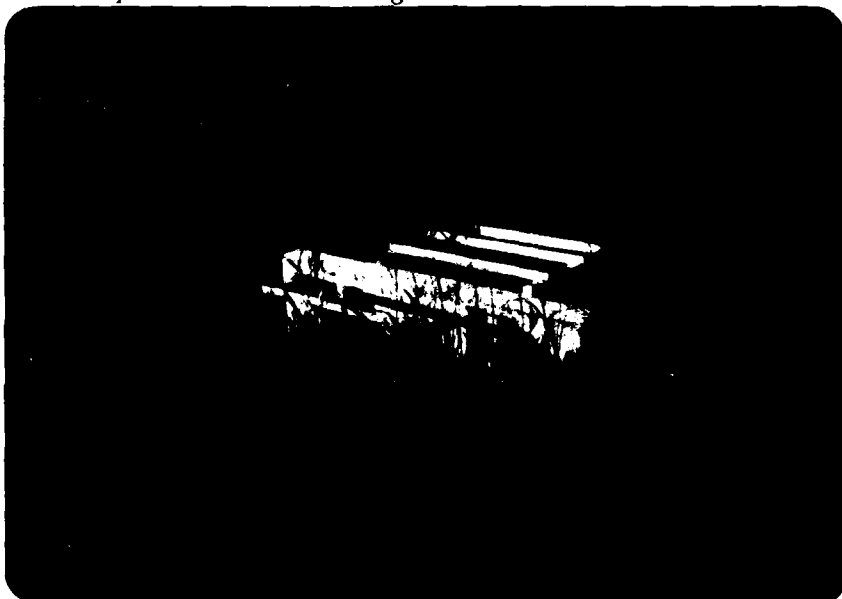
IN	MAXIMUM STORAGE	MAXIMUM OUTFLOW	DURATION OVER TOP	TIME OF MAX OUTFLOW	TIME OF FAILURE
	AC-FT	CFS	HOURS	HOURS	HOURS
	403	114	0.00	18.08	0.00
	479	101	0.00	16.08	0.00
	489	1487	0.33	16.00	0.00
	495	2017	0.50	15.92	0.00
	501	2472	0.67	15.92	0.00
	506	2879	0.83	15.92	0.00
	535	6744	4.08	15.83	0.00

Sheet 7 Appendix C

APPENDIX D



Top of Dam--Looking From South Abutment



Riser Structure--Primary Spillway





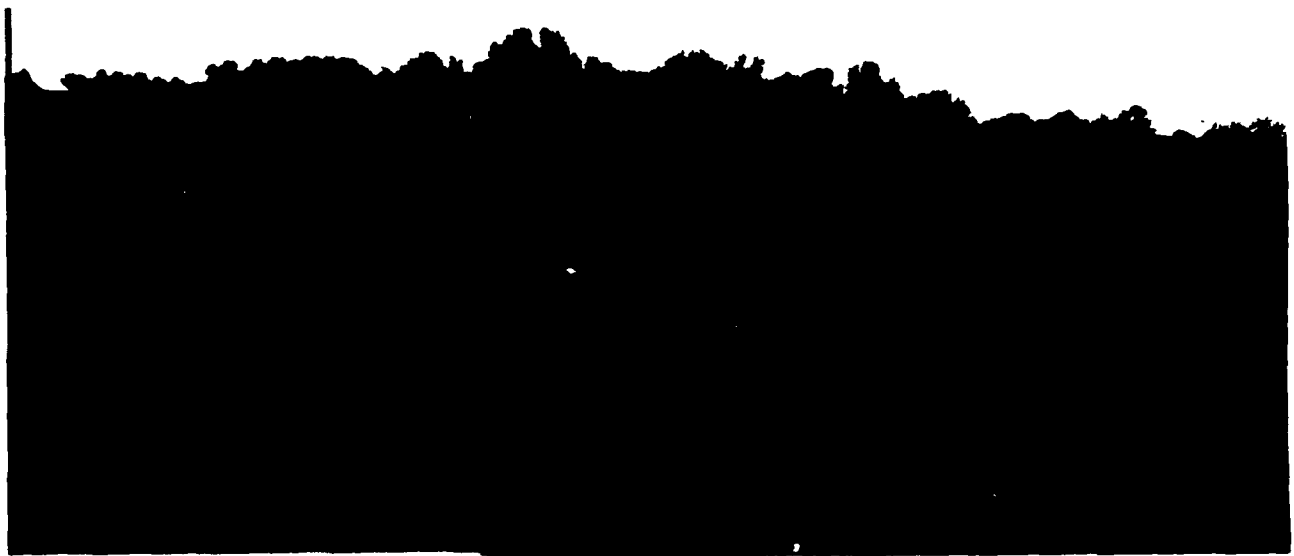
Outlet Pipe--Primary Spillway; Note Abutment Drain



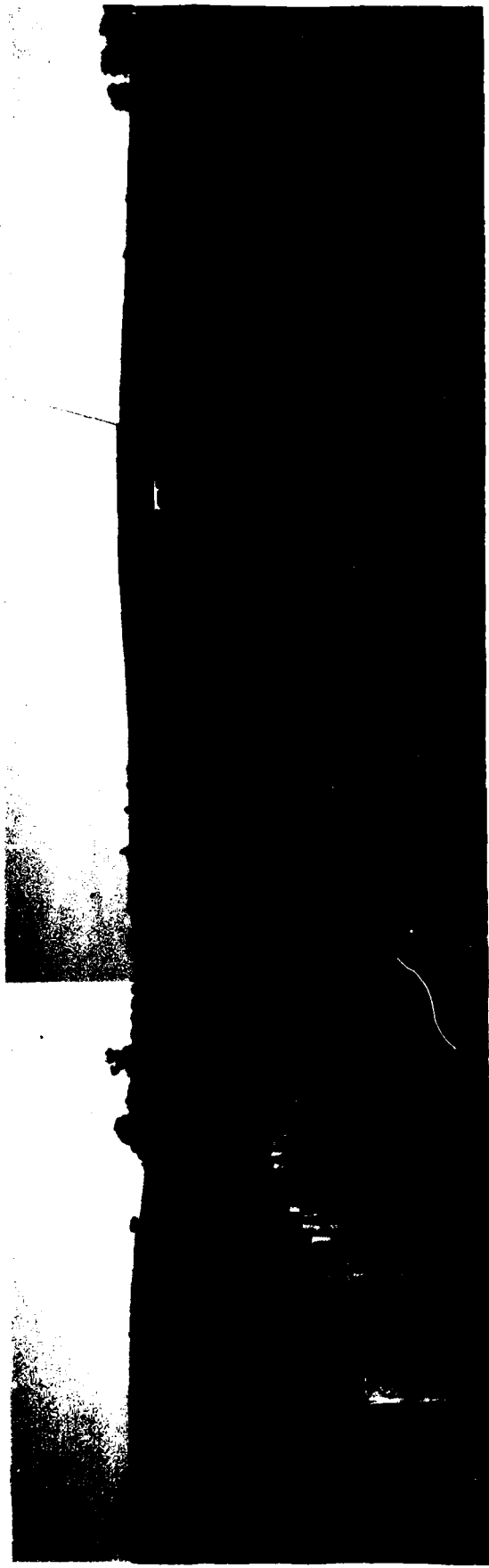
Outlet Channel--Primary Spillway



Downstream Side of Dam at Upper Berm Level--Looking North



Emergency Spillway--Looking Downstream From South Abutment



Lake and Watershed--Looking Southeast From North Abutment

Sheet 4 of 4  
Appendix D

END

DATE  
FILMED

12-81

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